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ALLOWABLE LEAKAGE IN CAST IRON PIPE[1]

BY CHARLES C. HOPKINS[2]

Until about the year 1887 leakage tests of cast iron water piping systems having leaded joints had been very few. The opinion prevailed that, if leaks did not show at the surface of the ground, there were none, or at any rate they were of negligible amount and systems were accepted without question as to their water tightness.

In the summer of 1887 the writer had occasion to repair a system constructed, without inspection or engineering advice or superintendence, by a contractor who wished to do satisfactory work and so instructed his workmen. The system consisted of about 6 miles of cast iron mains of 4 to 8 inches in diameter, the weighted average size being about 6 inches. The water pressure varied from nothing at the reservoir to about 70 pounds per square inch as a maximum, with a mean of about 50 pounds. The system was leaking at the rate of 5000 gallons per mile per day. The quantity of lead used in the joints was ample for good work. The leading and caulking of the joints were poorly done, and practically the entire leakage was through the joints. The leaks were so scattered over the entire system that they did not show at the surface of the ground, and none were very large. After some weeks spent in searching for the leaks and repairing those found, the leakage was reduced to 1000 gallons

[1] Presented before the New York Section meeting, May 3, 1929.
[2] Consulting Engineer, Rochester, N. Y.

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per mile per day. The writer then considering the system reasonably satisfactory, wrote to the late Charles B. Brush, outlining the conditions, but not giving the leakage either before or after repairs, and asked his opinion as to what he would consider a reasonable allowance for leakage. Mr. Brush was the only one known to the writer who had made careful leakage tests up to that time, and he replied that he would not consider 5000 gallons per mile per day excessive. The municipal authorities refusing to accept the system with one-fifth of that leakage, it was then gone over again, most of the joints uncovered and repairs made where necessary, until the leakage was reduced to slightly less than 200 gallons per mile per day, or about 33 gallons per inch mile per day, in which condition the system was accepted.

The foregoing experience and the fact that a leak of 50,000 gallons per day, or even much more, may never appear at the surface of the ground, convinced the writer and his former associates that specifications for a piping system should contain a provision as to the leakage permissible under the conditions obtaining for that system, and, commencing in 1889, such a clause was inserted in their specifications wherever it was practicable or not too inconvenient to make a test. The tests were generally limited to those on new complete systems, and consequently on the smaller piping. It is generally quite easy to design such systems so as to have convenient means for accurate measurement, that is, at least as accurate as measurements by pump displacement or meter. In the fifteen tests to be given later on, the leakages were determined from loss from or drop in stand pipes, elevated tanks, screen pots or other receptacles of known dimensions. Tests of other systems and pipe lines had been made from time to time by the writer's associates, but are not available, but the results obtained were substantially as good as those given.

Before giving consideration to the fifteen tests made by the writer, it seems desirable to consider the results of tests as are at hand and have been made on other pipe lines and piping systems. The leakages as found and allowable are computed in gallons per mile per inch of nominal pipe diameter. For the purpose of better comparison, the tests are arranged in order of size of pipe. Some few published tests are omitted where the leakage was excessive. It is not to be assumed that all published tests are collected in table 1, but the range covers pipes from 48 to 4 inches in diameter. Where the diameter

of pipe is given in a fraction of a standard size, it represents the weighted average diameter of the several sizes tested.

Inasmuch as the foregoing pipe lines were laid under varying conditions, such as different workmen, different inspection, different specifications, different soil conditions, etc., it naturally follows that there would be difficulty in arriving at any relation of leakage to the pressures to which the pipes were subjected. It is to be noted that the leakages given in table 1 vary from less than one to 950 gallons per inch mile per day, and the recorded pressures from 25 to 152 pounds per square inch, without any apparent relationship of leakage to pressure. Nevertheless, it seems clear that leakage must vary as the square root of the pressure, all other conditions being the same, but inasmuch as greater care in jointing is usually taken as pressures increase, this fact may account for the apparent non-relationship of leakage and pressure.

The tests where the allowable leakage is given show that only two exceeded that allowable, with a weighted average result of about 52 per cent of the weighted average allowable leakage, while the weighted average of the tests where the allowable leakage is not given or known to the writer is about 80 per cent more than the weighted average of the allowable leakages and 245 per cent more than the weighted average obtained where there was an allowable leakage. This in itself should be an argument in favor of specifying a leakage below which a piping system would be satisfactory.

An inspection of table 1 shows that results were obtained of less than 100 gallons per inch mile on the 42- and 20-inch lines at Hartford, reported by Mr. Saville, on the 20-inch line of the Metropolitan Water Works, reported by Mr. Killam, on the 18-inch line at Corpus Christi, on the Akron, O. 4- to 30-inch lines and on the Grand View Heights system, reported by Mr. Bradbury, and on the 6- to 10-inch lines at Akron, laid by the water superintendent and reported by Mr. Bradbury. As these cover 54 miles of pipe of different sizes and were laid under different conditions, it would appear that substantially as good results should be generally obtained. The weighted average size of these pipes is about 17 inches and the weighted average leakage per inch mile per day is 84 gallons.

Other tests than those given in table 1 have been made and published, but the data at hand have been insufficient from which to draw conclusions as to leakage per inch mile on new mains.

It is of interest to note what several engineers specify or consider

TABLE 1

Data on leakage in cast iron mains

PIPING SYSTEM	MILES OF PIPE	SIZE	AVERAGE PRESSURE	LEAKAGE, GALLONS PER INCH MILE PER DAY		AUTHORITY
				Measured	Allowed	
		inches	*pounds*			
Metropolitan Water Works	22.1	48		407		Brackett
Detroit, Mich.	2.82	46.99		280		Fenkell
Metropolitan Water Works	2.22	42		314		Brackett
Hartford, Conn.	0.50	42		115	230	Saville
Hartford, Conn	0.07	42		298	230	Saville
Hartford, Conn	1.56	42		32.8	230	Saville
Hartford, Conn.	1.79	42		79.6	230	Saville
Hartford, Conn.	1.46	42	99–135	231	230	Saville
Hartford, Conn.	1.34	42		0.97	230	Saville
Hartford, Conn.	0.11	42		144	230	Saville
Hartford, Conn.	0.66	42		43.6	230	Saville
Hartford, Conn.	7.49	42	120	87.9	230	Saville
Metrŏpolitan Water Works	4.83	36		425		Brackett
Columbus, O.	0.79	36	110	469	528	Gregory
Columbus, O.	0.81	36	110	422	528	Gregory
Metropolitan Water Works	1.38	30		106		Brackett
Akron, O.	2.73	30	66–130	82	200	Bradbury
Akron, O.	0.63	24	104–122	69	200	Bradbury
Metropolitan Water Works	3.90	24		461		Brackett
Hackensack Water Works.	11.00	24	110	265		Brush
Metropolitan Water Works	7.60	20		950		Brackett
Metropolitan Water Works	0.90	20	35–100	69		Killam
Hartford, Conn	0.44	20	93	15	230	Saville
Akron, O.	1.58	20	90–136	69	200	Bradbury
Corpus Christi, Tex.	14.1	18		89	144	
Akron, O.	1.69	16	85–150	135	200	Bradbury
Metropolitan Water Works	9.46	16		627		Brackett
Akron, O.	16.76	13.94	66–152	83.4	200	Bradbury
Akron, O.	1.84	12	77–150	102	200	Bradbury
New York City	28.1	12		615		N. Y. Report
Hartford, Conn.	0.88	12	106–125	116	230	Saville
Hartford, Conn.	2.07	11.85	93–125	186	230	Saville
Akron, O.	0.97	10	107–140	81	200	Bradbury
Akron, O.	0.54	10	75–105	133		Bradbury
	0.66	10	42.5	113		Williams. Laid in '94, tested '96

TABLE 1—*Concluded*

PIPING SYSTEM	MILES OF PIPE	SIZE	AVERAGE PRESSURE	LEAKAGE, GALLONS PER INCH MILE PER DAY		AUTHORITY
				Measured	Allowed	
		inches	*pounds*			
Akron, O	1.30	8	66–132	42	200	Bradbury
Akron, O	1.22	8	105–140	63		Bradbury
Hoboken, N. J	22.37	8		160		Brush
Grand View Hts. O	0.55	7.42		0.31		Bradbury
Medway, Mass	11.49	7.40	75	244		Smith
Akron, O	8.9	6.52		61.7		Bradbury
	4.79	6.42	50	157		Williams. Laid '93–'96, tested 1896
Akron, O	5.88	6	66–150	66	200	Bradbury
Akron, O	7.1	6	75–150	59		Bradbury
Englewood, N. J	4.78	6	25–100	226		Brush
Kinkaid, Ill	5	5*		470		
Akron, O	0.13	4	90–140	23	200	Bradbury

* Average.

permissible leakages based upon their experiences. These are shown in table 2.

The data given in tables 1 and 2 were obtained largely from the following:

Transactions A.S.C.E., vols. XIX, XXXIV, and XXXVIII.
Journal N.E.W.W.A., September, 1914, and March, June and December, 1916.
Water Works Handbook, by Flinn, Weston and Bogert.
Engineering News, Record and News-Record files.

In table 3 the results are shown of tests on fifteen pipe lines or systems, made by the writer. Tests of other lines and systems have been made from time to time by the writer and his associates, the results of which are not available, but, except in one instance, they were substantially as favorable as those given in table 3. They have been made on pipes up to and including 30 inches in diameter. The one instance referred to where the test was not favorable was made about thirty-five years ago on a system of piping of 4- to 10-inch size, the leakage being 450 gallons per inch mile per day, the weighted

average size being 6.22 inches. Owing to the lateness of the season when the system was completed, the fact that no leaks showed at the ground surface, and the abundance of the water supply, the municipal authorities accepted the system with the leakage as determined.

Generally the specifications admitted a leakage of a certain number of gallons per day per mile of pipe under the maximum static pressure, and if the leakage was found to exceed the amount specified, the contractor had the option of forfeiting to the municipality a fixed price per 100 gallons for the excess leakage or of reducing the leakage to that allowable. This price was usually placed at somewhat near the capitalized charge for which the municipality would be willing to sell water to the average consumer. In some cases, if the total leakage exceeded about 250 gallons per inch mile per day,

TABLE 2

	SUGGESTED ALLOWABLE LEAKAGE, GALLONS PER INCH MILE PER DAY
E. G. Bradbury in 1912	60–250
King on 6-inch pipe	83
C. F. Loweth	60–80
E. G. Bradbury in 1914, trench testing	100
Barbour and Bradbury at Akron, O.	200
J. H. Gregory, 20 to 30-inch at Columbus	507–528
Burns and McDonnell	80
N. Y. Aqueduct Specifications	About 240

it was made obligatory for the contractor to reduce it to within that amount before he could exercise the right of option referred to.

The tests on these systems were all made within a few days after the entire completion of the pipe laying. On only one, number 14, was any of the systems tested before the trenches were backfilled. In this instance the contractor for his own satisfaction decided to test the various sections and correct any leaks before backfilling, although he understood that that would not make certain that there might not be leakage found on the official test. This was number 14 where the official test showed a leakage of 39 gallons per inch mile. Number 13 was the only system on which uncovering of the pipes was done except where there was leakage known by its showing at the surface of the ground. As this system showed a leakage largely in excess of

that allowable, the contractor after testing the system by sections and thus determining in which ones the trouble lay, did uncover considerable portions of the several sections and repair the leaks found before the final test as given in table 3.

Open trench testing with correction of defects before backfilling is desirable wherever the cost and inconvenience occasioned by so testing are not too great. It will not, however, insure a bottle tight pipe line, but is usually a good guarantee that no serious defects

TABLE 3

Results of further leakage tests

PIPING SYSTEM NUMBER	MILES OF PIPE	SIZE	AVERAGE PRESSURE	LEAKAGE, GALLONS PER INCH MILE PER DAY	
				Measured	Allowed
		inches	*pounds*		
1	0.53	24	75	174	125
2	0.75	12	78	38	50
3	0.58	10.41	50	79	38
4	5.03	6.86	52	17	58
5	8.25	6.53	78	145	61
6	4.74	6.44	52	62	62
7	4.08	6.35	52	53	63
8	5.50	6.35	71	42	78
9	3.03	6.29	48	63	63
10	3.38	6.26	54	48	64
11	4.62	6.02	70	30	33
12	4.44	5.89	82	70	85
13	2.41	5.73	76	56	70
14	5.43	5.42	56	39	73
15	4.23	5.32	87	109	94
Averages			65.4	68.3	67.8
Weighted averages				66.6	66.6

exist, and should result in less leakage than any specified permissible leakage being obtained after the trench is backfilled and a final test made. It should not take the place of a final test, since expansion and contraction of the pipe may cause slight leaks and uneven settlement of the pipe due to the backfilling may also cause leaks and even cracked pipe.

The writer believes that leaks made up of many slight ones will tend to cease eventually, but that, if the individual leaks are large,

they will increase with time, but so far as the writer has had an opportunity to observe there has been no appreciable increase in the leakage in course of time where good workmanship was made necessary under specifications allowing a reasonable leakage.

Specifications for pipe lines should provide for damages for exceeding allowable leakages, and also give the contractor performing the work a bonus for leakage less than the allowable. All leaks, the location of which are known, should be corrected before testing the pipe. The damage clause should not become operative until the contractor has reduced the leakage below an amount stated in the specifications, to which amount the contractor should be required to reduce it. It is also well to specify that the leakage from no one section of a system of pipes should exceed a fixed multiple of the average on the entire system.

In conclusion, the writer is of the opinion that for pressures not exceeding about 65 pounds (equivalent to 150 feet head) and for pipe sizes of 10-inch and under, an allowance for leakage of not to exceed 65 gallons per inch mile per day should be easily obtained, and that for sizes greater than 10-inch the above allowable leakage should be increased somewhat, but not to exceed 25 per cent, unless there are extraordinary conditions that might affect the quality of the workmanship. For pressures exceeding 65 pounds the allowable leakage should be increased on all sizes to something less than the ratio of the square root of the head. The writer suggests the cube root of the head or even less, inasmuch as greater care in jointing is most likely to be taken under large than small pressures. There would then be allowed on the smaller sizes for 130 pounds, or double the head or pressure, 82 gallons, and, on the larger sizes, up to 102 gallons for 130 pounds pressure.

THE MAINTENANCE AND OPERATION OF GATE VALVES AND FIRE HYDRANTS

By Carl A. Hechmer[1]

It is surprising, especially in small towns and villages, how little attention is given the maintenance of gate valves and fire hydrants on their water systems. The need of good engineering advice and design is recognized and is compulsory in almost every state. Practically every state health department has a law compelling the submission of plans for approval before a water system can be built. The question of good materials is also generally given the utmost consideration and the best valves and hydrants obtainable, in the judgment of those in charge, are purchased and put into the water system. In many instances they are promptly forgotten until an emergency arises and quick, sure operation is necessary. Unless valves and hydrants are given regular attention and properly maintained, the water works superintendent has no assurance that they will function properly. This fact applies to any make of valve or hydrant or to any other mechanical device or piece of machinery. Unfairly the failure reflects on the designing engineer or the manufacturer of the material and the local department is given a clean bill of health.

VALVES

All valves on a water distribution system should be inspected once each year. This is the minimum requirement of the Fire Underwriters' Association. However, semi-annual inspection is advisable, if finances will allow it. Not only will the mechanical condition of the valve be assured, but valve boxes covered by paving material or earth will be located and reset ready for immediate operation in emergency. This is especially true in rural and suburban sections with a large percentage of unpaved streets. When a large leak occurs, excitement prevails and considerable time may be lost locating a valve box. Occasionally a large stone or stones are found down in

[1] Department Engineer, Washington Suburban Sanitary District, Hyattsville, Md.

valve boxes and these conditions are revealed on inspection and removed. In case the valve has to be dug up, uncovering the valve, it is advisable to install new packing in the packing gland which will prevent digging it up for repacking for a long time. Always repack a valve when it is uncovered for any repairs. Packing is cheap and easily installed while the valve is exposed. To dig up a valve just to stop a packing leak is an expensive operation. Inspection also brings out valves which were carelessly or accidently left closed. Any water distribution system is weakened by gate valves being closed when they should be open.

In addition to the box inspection, a valve inspection should proceed as follows: First set the valve key on the valve nut preparatory to turning the stem. Pour a small quantity of kerosene or diluted automobile crank-case drainings down the valve key stem. The oil will run down the key to the operating nut, thence to the valve stem and lubricate the valve stem against the packing gland. Finding its way into the packing gland, the packing will be softened and revived. Then operate the valve up and down several times, bringing the gate down into the groove hard each time and then opening the valve about one-quarter way. Valve troubles, preventing complete shut down, are usually caused by the accumulation of sediment under the seat. Each time the gate is pushed down into the groove a small portion of this sediment is pushed out the sides. The high velocity of the water through the partly opened valve removes what is pushed out and the operation is repeated until the groove is sufficiently cleaned to allow the gate to seat properly. This procedure is especially recommended when valves fail to hold when attempting a shut down. Of course, if the groove is filled with large stones or jointing material, the valve must be dug up and taken apart for repairs. The writer once operated a 16-inch valve up and down thirty times and secured a perfectly tight shut down. The valve had not been operated for a long time, having been declared worn out, since it always allowed a large volume of water to pass with the gate turned down hard with levers on the key.

In operating the valve a check should be made of the number of turns on the stem. Generally for the same size and make of valve the number of turns to close or open the valve completely is the same and a check can be made to see if the stem is bent preventing the valve from completely closing or opening. Either condition should

be corrected, as partly open valves weaken the system and, obviously, partly closed valves will not shut off the water.

If the valve stem can be turned around and around with no apparent effect on the gate, the stem is broken and must be repaired.

Geared valves, usually set in manholes, must be given special attention on inspection. The gears should be cleaned with a wire brush and greased with a light clinging grease. The stuffing box should, of course, be examined and the by-pass valve should receive the same attention as the smaller valves on the distribution system.

FIRE HYDRANTS

Proper care in the installation of a fire hydrant is necessary and makes maintenance less expensive. Proper drainage of the hydrant barrel after the main valve is closed is essential to prevent freezing of the hydrant in northern climates. The best method is to provide a bed of crushed stone or gravel around the bottom of the hydrant. The amount of stone necessary is dependent on the nature of the soil. In loose sandy soils a much smaller drainage bed can be provided than in a clay soil that absorbs water very slowly. A safe margin is to provide sufficient area to allow the drainage of an amount of water equal to twice the contents of the hydrant barrel. Connections are often made direct from the hydrant drain valve to the sewer. However, there are two serious objections, first the cost and second the direct connection between a sewerage system and a domestic water supply, even though the possibility of contamination reaching the water in the mains is remote. Generally water mains are laid on one side of the street and sewers on the other. Hydrants are installed at the curb on the same side as the water mains to shorten the length of the lead. To connect each hydrant drain across the street to the sewer would prove much more expensive than providing a stone drainage bed.

Fire hydrants must be inspected at least twice each year, in the spring and fall. Four inspections annually should be made, if possible, and this number of inspections is advocated by the Fire Underwriters' Association. In high value districts, weekly inspections are often made during extremely cold weather. Arrangements should be made between the water department, the fire department and any other possible users of hydrants to report to the water department immediately after using a hydrant during freezing weather and an immediate inspection made to insure the good condition of the hydrant.

Use of hydrants by private individuals or contractors should be allowed only by special permit and the daily charge made high enough to permit daily inspection during freezing weather. Dur ng mild weather the inspection charge can be reduced as weekly inspections are sufficient.

Valuable data may be secured by the fire hydrant inspection by measuring the static pressure, the residual pressure with one nozzle open and the flow of water from the hydrant. Weak points on the distribution system can thus be determined and improvements due to new installation readily noted. System weakness due to unknown closed valves is also discovered and can be corrected.

The following procedure is suggested for making a hydrant inspection:

First make a physical examination of the hydrant, noting the condition of the operating nut, the nozzle caps and chains and the general appearance of the hydrant. Then listen for leakage with an aquaphone placed on the operating nut. Then with the nozzle caps on tight open the hydrant valve. If static pressure readings are to be taken replace one of the nozzle caps with one connected to a pressure gauge. Note if all the nozzles are tight in the hydrant and whether the drain valve has closed properly. If the drain valve has not closed water will appear around the barrel of the hydrant on the outside. Then close the main hydrant valve and take off one of the nozzle caps. Observe the rate of drainage of the barrel to determine whether any obstruction has entered the drain valve. To clean the drain valve replace the nozzle cap and crack the main valve. As it requires about three full turns open on the main valve to completely close the drain valve on most makes of fire hydrants, the latter is still open on two turns. The full pressure of the water in the hydrant generally will push out any obstruction in the drain hole and the jet effect of the water through the hole will cut away the stone or earth which has packed around the outside. This simple remedy will eliminate the necessity of digging up the hydrant in most cases to correct drainage troubles.

With one nozzle cap off open the hydrant and thoroughly flush out the water in the lead. At this time the observation of residual pressure and rate of flow should be made. Then close down the hydrant slowly. If the hydrant fails to close off entirely, the trouble is probably being caused by an obstruction under the main valve.

Do not put extra leverage on the hydrant wrench, but open the hydrant up again and then close it, repeating the operation several times. Do not attempt to close down beyond the point where the obstruction prevents the main valve from seating properly, as the obstruction will become embedded in the main valve and cannot be flushed off, or the valve seat will be damaged. Both valve and valve seat are damaged by attempts to force closing against obstructions. Foreign matter can generally be dislodged by flushing and it is then not necessary to dismantle the hydrant.

After the hydrant is closed and the rate of drainage checked, the stem should be oiled and the nozzles greased with a graphite grease. This kind of grease will not wash off and it prevents the nozzle caps from sticking.

Record of the inspection should be kept on a field sheet. The location of the hydrants to be inspected can be entered on the sheet in the office. These will allow correct routing and prevent skipping any hydrants. The information on the field inspection sheets can then be transferred to a permanent record card in the office. A card should be in the file for each hydrant, stating the size, make of hydrant and information regarding pressure at the hydrant. The date of each inspection should be entered on the card together with the observed pressures and rate of flow. Necessary repairs and their costs can also be recorded on the card and a history of each individual hydrant is easily obtained.

For quick reference the following is suggested: Use a map of the water system mounted on wall board with a push pin to denote the location of each hydrant. The color of the pin can show the condition of the hydrant as reported daily by the hydrant inspectors and the repair foreman. Use a blue pin for a hydrant in good condition, a black pin for a hydrant in need of minor repairs but not out of service, and a red pin for a hydrant out of service. Then when an inspector reports a hydrant out of service it is possible to tell at a glance whether the hydrants around it are in good condition. If several makes of hydrants are in use it is an advantage to know the make of hydrant before sending out a repair gang so the proper parts can be carried along. By using a small round colored disc under each push pin it is possible to determine at a glance the make of each hydrant, a different color being used for each make of hydrant.

For the sake of appearance and to protect the metal, hydrants

should be painted once every two years and if possible annually. Use a good grade of paint and care should be exercised to prevent getting paint on the nozzle threads or operating nut. The grooves on the nozzle caps which hold the chain links must be thoroughly brushed out as excess paint will harden and retard the fire department in removing the nozzle caps to connect fire hose.

THE DESIGN OF THE COBBLE MOUNTAIN DAM[1]

By Allen Hazen[2]

This paper is limited to the design of the Cobble Mountain Dam because the dam is not yet built. The story of a dam cannot be adequately told until the construction work has been completed and the water has reached the spillway level. Sometimes the story must wait until maximum floods have occurred.

The Cobble Mountain Dam is being built by the City of Springfield, Massachusetts, as part of a development of its water supply. Mr. E. E.Lochridge as Chief Engineer is in charge of the entire work. Mr. H. H. Hatch is Resident Engineer for the construction of the Dam and Tunnels. Hazen and Whipple, (now Hazen, Everett and Pirnie) the writer's firm has prepared the plans and has acted as Consulting Engineers. Mr. Charles T. Main of Boston acted as special Consulting Engineer, checking the general plans and estimates, and Mr. George A. Orrok has aided in connection with the water power and electrical development.

There have been four principal contracts:

1. The Diversion Tunnel, let in July, 1927, to Coleman Brothers, of Boston and completed about a year later.
2. The Main Dam, Spillway and Reservoir was let to Winston and Company of Kingston, N. Y., in July, 1928, and
3. The Cobble Mountain Tunnel to Frazier-Davis Construction Company, of St. Louis, at the same time.
4. The surge tank, penstock, power house, and electrical equipment to Turner Falls Power and Electric Company.

Some road and other work has been carried out by force account. The buildings over the shafts and some small parts of the work are not yet under contract. It is the intention that the whole works shall be completed by October 1, 1930.

The Cobble Mountain Dam is larger and will cost more than could have been built at this time from water funds at present water rates

[1] Presented before the Toronto Convention, June 26, 1929.

[2] Consulting Engineer, New York, N. Y.

FIG. 1. SITE OF DAM

and with a balanced budget. It is the auxiliary power that makes it financially possible now. The reservoir is very high in elevation and there was an opportunity (recognized when this source was first proposed in 1905) of developing water power when full storage became available.

After extended study in the last years, a contract was entered into with the Turners Falls Electric and Power Company, under which the Company joined with the City in carrying out the development and will pay an annual rental for thirty years for the privilege of generating power.

DESCRIPTION OF DAM

The dam is of the hydraulic fill type, with a top width of 50 feet, a freeboard of 20 feet above spillway level, and 13 feet above the top of the proposed 7 foot flashboards. The slopes are moderate and become flatter toward the bottom of the dam.

Rock fill will be a feature of the dam, a large block of this material being placed in each of the toes, rising to a total height of about 110 feet above stream bed. Part of this rock fill was required by the original design, but after the contract was let the amount was increased by arrangement with the contractor as permitted by the specifications under the unit prices of the contract.

As a finish to the downstream toe there will be a concrete arch with a radius of 100 feet cut securely into solid rock on each side, and resting without being cut into the rock on the bottom, with spaces for drainage underneath. This arch is 35 feet high and from 14 to 20 feet thick and cuts off what would otherwise have been an extreme extension of the downstream toe.

The shape of the dam is unusual. The gorge is narrow and curved. The side slopes are approximately 37° or one on 1.33. The bottom width is something like 70 feet. Under these conditions the dam has a length along its crest of only 700 feet, but is 1505 feet wide on the bottom measured along the thread of the stream. It is therefore more than twice as wide as long.

The height of the dam measured from the stream bed to the spillway level is 215 feet with 7 feet additional to the top of the proposed flashboards. To the roadway level on the top of the dam it is 235 feet. The cutoff trench in rock will extend 10 feet or more below stream bed and is not included in these heights.

At the proposed flashboard level, elevation 952, the Cobble Moun-

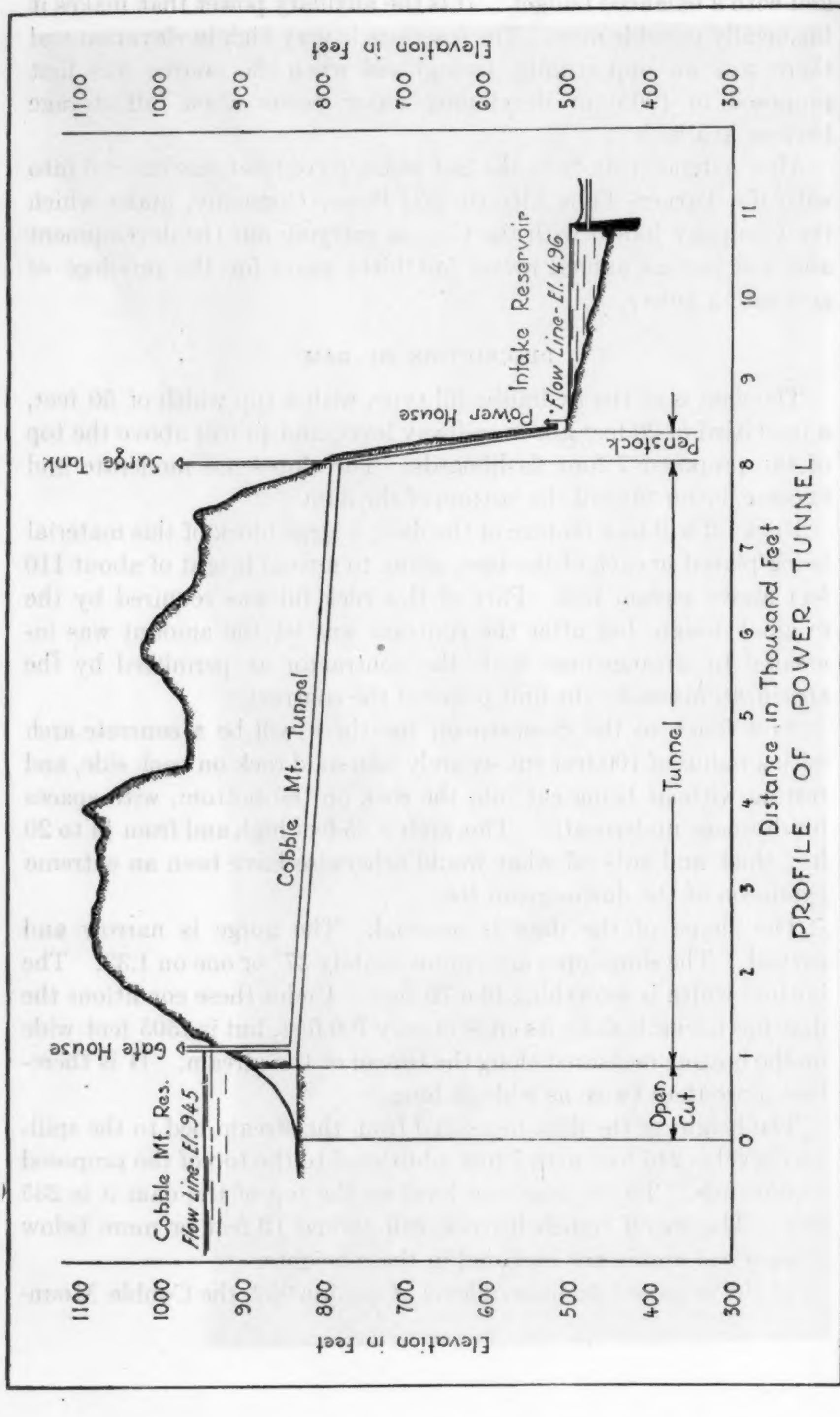

Elevation in Feet
1100
1000
900
800
700
600
500
400
300
Cobble Mt. Res.
Flow Line El. 945
Gate House
Cobble Mt. Tunnel
Surge Tank
Power House
Intake Reservoir
Flow Line-El. 496
Open Cut
Tunnel
Penstock
0
1
2
3
4
5
6
7
8
9
10
11
Distance in Thousand Feet
PROFILE OF POWER TUNNEL

FIG. 2

tain Reservoir will have an area of 1120 acreas and will contain 22,287 million gallons of water. Of this amount 2021 million gallons below elevation 840 are not available for power, but may be drawn in case of need for water supply.

In addition to the storage in the Cobble Mountain Reservoir the Borden Brook Reservoir, constructed in the year 1909, on a tributary stream, has a capacity 2,500 million gallons. Of this, under the arrangement with the Power Company, 1,500,000 gallons may be drawn for power and 1,000 million gallons are held as an emergency reserve for water supply.

The volume of Cobble Mountain dam is approximately 1,800,000 cubic yards, of which something like 400,000 cubic yards will be rock fill and the remainder earth. All but a small part of the latter will be placed hydraulically.

Because the stream was not straight, the lower parts of the toes are turned on a regularly graduated arrangement so that the contours of the finished dam, in a general way, will be perpendicular to the sides of the valley. The total curvature of the valley is some 90° from one toe to the other and of the dam 60°, leaving 30° at a sharp bend in the stream near the upper toe. The stream bends to the left and this procedure leads to steeper slopes on the faces of the dam along the left bank of the stream and flatter slopes near the right bank.

The procedure in general was to make the desired standard slope, selected in advance, apply approximately on the side where the slope was steepest and to leave a flatter slope, or, in one case, a berm near the right bank of the stream.

The material in the damsite is mica schist rock covered to an average depth of about 4 feet, with soil. The soil is mixed with fragments of mica schist, with here and there a sprinkling of harder rock of glacial origin. This cover is more or less uniform over the entire site with the exception of some places where all surface material had slid off into the stream leaving bare rock exposed in great ledges.

The character of the mica schist is known from various excavations in the neighborhood and especially from the experience in driving the diversion tunnel completed in 1928. The first hundred feet or so at each end of the tunnel is more or less broken and seamy and carries a little water. Otherwise, the rock is solid and free from seams. The seams, except those near the surface, carry but little water and the rest of the rock none at all.

To prevent seepage through seams in the rock where the core of the dam is to join the rock, a cutoff trench will be cut across the bottom and sides of the valley to a depth to be determined as the work proceeds. In the bottom of this trench will be dug narrower trenches in which concrete walls will be built extending 3 feet above the bottom of the wider trench. One of the walls will be carried to the top on each side of the valley.

The rock below the bottom of the trench is to be drilled and grouted to cut off seams and make the rock watertight. The clay core will fill the rock trench and cover the concrete cutoff walls and make a watertight seal.

The material for the rock fill is obtained from the mica schist in the immediate neighborhood of the dam. The rock rises on each side at the damsite to about 470 feet above the stream bed or 230 feet above the top of the dam, affording opportunities for convenient and ample quarries on both sides. The rock excavated in the course of the work, including that from the spillway, is placed in the dam as rock fill.

The mica schist, although suitable for rock fill, will not crush to make concrete stock and there is no rock in the neighborhood of the work fit for masonry of any class. Williamsburgh granite found in some of the neighboring hills is not suitable for quarrying. There are quarries of excellent granite within 20 miles of the site, but the use of this material for masonry was not seriously considered. All material for both the concrete work and the hydraulic fill must be brought from pits about a mile away.

At the dam site the soil will be sluiced into the dam and there are some limited deposits of fine grained material not far away that may be useful in construction.

The borrow pits for the hydraulic fill are glacial deposits of sand and gravel. There are fine grained layers running down, in places, to rock flour. The deposits are extensive and contain many times the required volume of material. Selection will be necessary to secure the required grain sizes. Pits carrying different grades of material will probably require to be operated simultaneously.

The contractor proposes to excavate the material in the pits with steam shovels and to carry it on standard gauge railway track to the dam where it will be dumped from tracks on the top of the rock fills, or from trestles above them, to the toes of the dam. The material will then be sluiced into the dam.

Water for sluicing is to be provided by floating pumps on the pool in the center so that the water used for building the dam will be circulated and kept separate from the water of the running stream, and so avoid pollution of the water supply.

As the material from which the dam is to be built is sandy and gravelly in character, the problem of control of grain size is quite different from that in several other hydraulic dams recently constructed in which the percentage of clay has been high.

The specifications provide that the material in the central core shall have an effective size of about 0.01 mm. the intention being to secure a material fine enough to be for practical purposes, watertight, but, on the other hand, coarse enough and pervious enough to permit the core material to drain sufficiently for its own consolidation.

It will be remembered that in some hydraulic dams built in the past (Hydraulic Fill Dams, Am. Soc. C. E., vol. 83, p. 1713, 1920) the core material has been so fine that it was incapable of draining within any reasonable length of time and has remained permanently fluid in the center of the dam. In some cases the pressure of this fluid core has exceeded the resistance of the toes and has ruptured them during construction.

An absolute requirement of stability is, therefore, that the material should be sufficiently coarse grained as to drain and become solid and stable.

On the other hand, some hydraulic fill dams of sandy material have been so coarse grained and free from fine material that there has been an appreciable amount of leakage through them.

In the case of Cobble Mountain Dam approximate watertightness is desirable and necessary; to this end the core size must be regulated to insure that the completed work will be nearly watertight with only so much seepage as is necessary to insure drainage and stability.

In connection with this important subject a method of elutriation and microscopical examination has been devised to determine the character of the material being placed in the core as the work proceeds. This may well form the subject of a separate paper to be presented at some future time by some of the men who have had more particularly to do with this part of the subject.

DIVERSION TUNNEL

Before beginning construction of the dam itself, it was necessary to divert the water of Little River. Work on a diversion tunnel

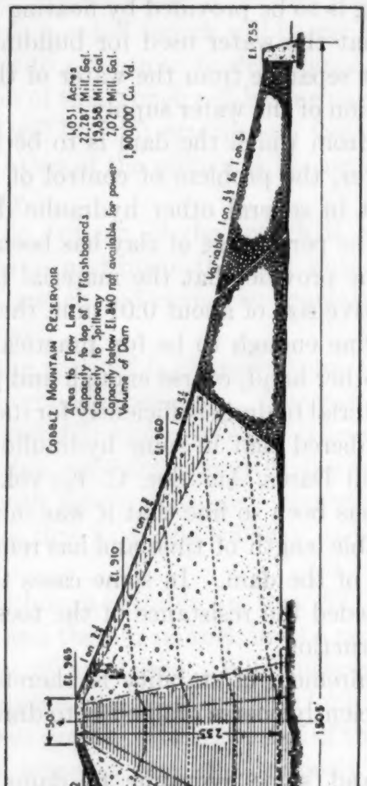


FIG. 3. COBBLE MOUNTAIN DAM

1620 feet long was started a year in advance. This tunnel is horseshoe shaped equal to a circle 12 feet in diameter and has a slope of 18 per 1000. Its invert is a few feet above ordinary water level at both ends.

In connection with this tunnel there was built a cofferdam of rock fill, as a preliminary operation, so placed that it would form part of the permanent upstream toe of the dam. This was faced with enough soil to secure approximate watertightness. This cofferdam had a height of 70 feet and was required to hold the water at a level that would produce a velocity in the tunnel to develop its capacity.

The tunnel was designed to have a capacity of 4,000 cubic feet per second, an amount comfortably in excess of any flood that had been recorded during the twenty years that the stream had been gauged. On November 3, 1927, four months after the contract had been let, there was a great flood. This was at the time of the Vermont flood. The maximum rate of discharge at the intake dam 3 miles below was 6100 cubic feet per second; and the average for twenty-four hours almost reached 4000 cubic feet per second. This was before the diversion of the stream.

If such a flood had occurred one year later it would have taxed to the utmost the storage in the reservoir above the cofferdam and the delivering capacity of the tunnel.

During the present season the dam will be raised to a point that will increase the storage back of it to a point where there will be no possibility of its being overtopped. This will occur before the possible storage capacity back of the dam becomes great enough to be a potential danger to the valley below.

It may be here mentioned that the catchment area of the dam is 45.8 square miles, with an additional area between it and the intake dam, so that the total catchment area at the waterworks intake is 48.5 square miles.

The mean flow at the intake dam is about 62 m.g.d. It will be proportionately less at Cobble Mountain Dam or about 58 m.g.d.

The diversion tunnel was lined with concrete and the space back of the concrete grouted. At a convenient place a shaft 218 feet deep was cut from the surface and a place selected upstream from it for the permanent closure and control works. In this place, after the dam is built, a concrete plug will be placed and grouted. Through this two 42-inch pipes will pass leading to gates and guard gates in duplicate with discharging capacity of more than 1000 cubic feet per

second, from which water may be drawn for water supply purposes or to lower the water in the dam in any emergency. Otherwise these gates will not be used.

The capacity of the outlets is such that using both tunnels water can be lowered about 3 feet per twenty-four hours, the rate of drop increasing as the water falls. The reservoir could be emptied in about thirty days.

THE COBBLE MOUNTAIN TUNNEL

The Cobble Mountain tunnel which will be the ordinary outlet and will supply water to the power house, is 7100 feet long and is equivalent in area and capacity to a circle 10 feet in diameter. It is to be concrete lined throughout and grouted. The entrance to this tunnel is 76 feet in elevation above that of the diversion tunnel. The sill at its entrance is at elevation 830, 122 feet below the top of the proposed flashboards.

This tunnel is driven through mica schist rock, similar in character to that penetrated by the diversion tunnel, and, for the most part, completely dry. A small amount of Williamsburg granite, a coarse grained local variety, was encountered. The tunnel location was selected to give ample rock cover to resist pressure, except in the lower 500 feet which are to be steel lined.

The control of water through this tunnel will be, ordinarily, by gates at the power house and otherwise by a gate at the inlet which can be used to hold the water back while the tunnel and penstocks are unwatered for inspection or repair. This gate is a Broome gate 12 feet wide and 13 feet high covering a carefully formed bellmouth.

A short section of tunnel upstream from this entrance is so arranged that when the water is drawn down to elevation 900 or lower, it will be possible to sink an emergency gate to cover the outer entrance and to permit the Broome gate and its seat and everything connected with it to be unwatered. The outer gate will not be provided at present, but the seat will be built so that this remedy may be applied at a future time if it should prove necessary.

At the lower end of the tunnel a steel pipe lining will be used extending to a point where the rock above is approximately at the elevation of high water in the reservoir. At its lower end this pipe is connected to a surge tank and then to three penstocks. These go down a steep declivity to the power house which is built at the upper end of the small reservoir formed by the intake dam. From the in-

take dam water is drawn to the filters as needed and the rest of the flow spills down the stream.

DEVELOPMENT OF POWER

The flow line of the Cobble Mountain Reservoir, at the flashboard level, will be 456 feet above the present flow line of the intake reservoir. This possible head on the wheels will be reduced by draw-down in Cobble Mountain Reservoir and by friction in the tunnel and penstocks. The working head will vary from about 450 feet down to about 350 feet, with a possible extreme minimum of 330 feet. It will probably average in the neighborhood of 400 feet.

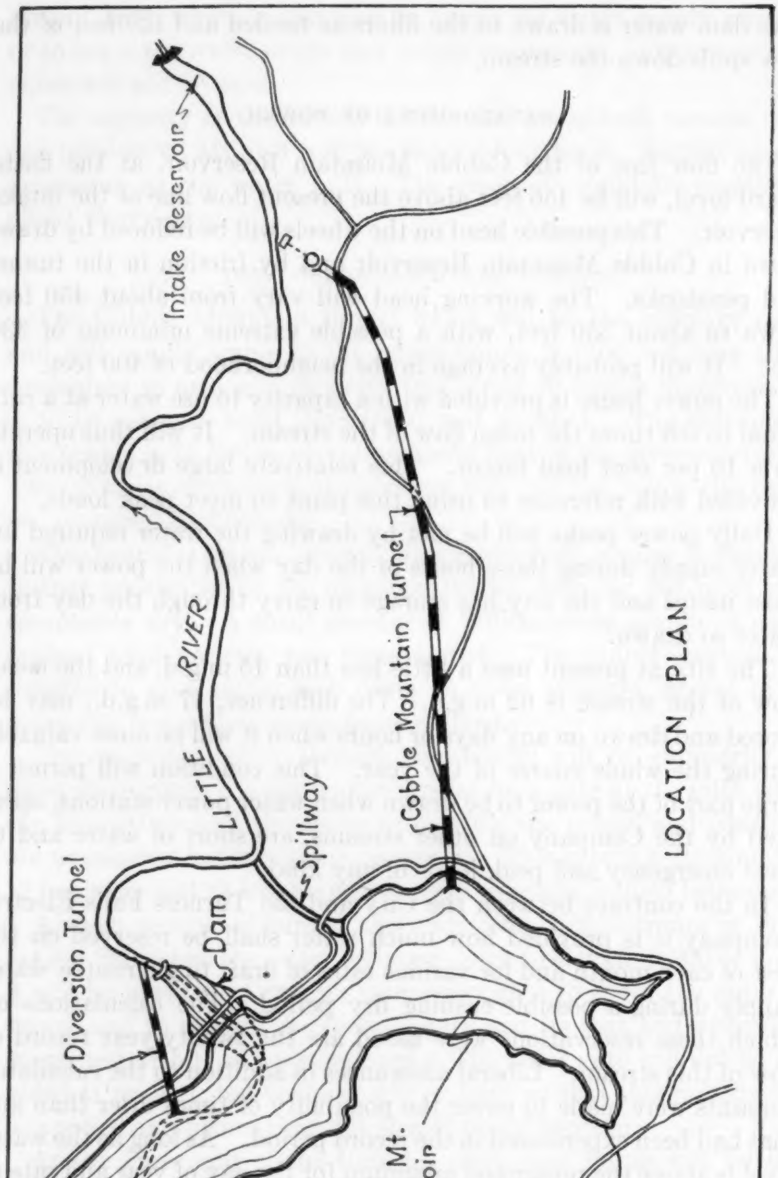
The power house is provided with a capacity to use water at a rate equal to ten times the mean flow of the stream. It will thus operate on a 10 per cent load factor. This relatively large development is provided with reference to using this plant to meet peak loads.

Daily power peaks will be met by drawing the water required for water supply during those hours of the day when the power will be most useful and the city has storage to carry through the day from water so drawn.

The city at present uses a little less than 15 m.g.d. and the mean flow of the stream is 62 m.g.d. The difference, 47 m.g.d., may be stored and drawn on any days or hours when it will be most valuable during the whole course of the year. This condition will permit a large part of the power to be drawn when water power stations, operated by the Company on other streams, are short of water and to meet emergency and peak loads of any kind.

In the contract between the City and the Turners Falls Electric Company it is provided how much water shall be reserved on the first of each month and for various rates of draft to guarantee water supply during a possible ensuing dry period. The calculations on which these reservations were based are the twenty-year record of flow of this stream. Liberal allowances in addition to the calculated amounts were made to cover the possibility of times drier than any that had been experienced in the record period. As long as the water level is above the prescribed minimum for the day of year and rate of draft, the Company may draw water as it pleases. When that limit is reached the Company's rights to draw cease; and it may then only draw the quantity of water required for daily water supply until there is again a margin.

The Company is to pay the city an annual rental of $270,000 for



LOCATION PLAN

Fig 4

the lease of the power and privilege of using the water. This annual rental is subject to a number of small adjustments and to a decrease of $20,000 per annum when the city's use of water exceeds 30 m.g.d. Other similar reductions for still greater possible future increases in supply are provided. The contract runs for thirty years which is the period of the longest serial bonds issued to pay for the works.

The Company builds the power house, penstocks and all electrical equipment for the city at cost, but not to exceed $1,100,000, which amount the city will pay to the Company on the completion of this part of the work.

FLOOD FLOWS

The spillway of a dam of this character is most important. When the work was started the largest flood on record measured at the intake dam was 4410 cubic feet per second. After the work was started there was a larger flood as noted above, with a maximum of 6100 cubic feet per second.

From a study of the records of flow of this particular stream, the indications are that the greatest average flow for twenty-four hours to be expected once in a hundred years would be 4850 cubic feet per second. On present evidence there is a 1 per cent chance of a flow of this kind in any one year, and, we may say, from general experience with flood records, a probable chance of about 0.1 per cent per annum of the occurrence of a flood of 8800 cubic feet per second. The greatest natural rate for shorter periods than twenty-four hours will not be important after the dam is built with its ample freeboard storage to absorb them.

In view of the character of the occupation of the valley below the greatest precautions were warranted against overtopping of the dam, and it was decided to lay out a spillway channel with a capacity of 20,000 cubic feet per second, equal to more than 436 cubic feet per second per square mile of tributary area. This is equal to five times the record twenty-four-hour average flood of the record period.

There was an excellent natural site for the spillway through a notch in the mica schist rock on the opposite side of Cobble Mountain from the dam.

Using this notch required a considerable amount of rock excavation, but the rock obtained could be placed and is being placed in the dam.

The crest of the spillway was laid out 135 feet long. The channel

from it drops 15 and narrows as a bellmouth to a width of 50 feet and then extends with a grade of 1 per cent for a total distance of about 700 feet from the entrance, at which point the rock falls off abruptly on a slope of approximately 31° to the river bed. The total vertical drop is about 255 feet. The flow will reach the river about 3000 feet downstream from the lower toe of the dam.

With a well-rounded crest and a gradual approach, the spillway should carry 20,000 cubic feet a second with about 13 feet head. The water level in the reservoir would then be at elevation 958, at which level it would still be 7 feet below the top of the dam.

The spillway channel was calculated so that at each point, with an assumed water level, which would pass 20,000 cubic feet per second, the water level, plus the velocity head, plus an estimate for friction from the entrance to that point would equal the assumed reservoir level, i.e., 958.

The allowance for friction not being capable of being estimated closely, a liberal and perhaps excessive allowance was made for it, amounting to 1 per cent of the total distance from the entrance to each point of calculation. Water velocities in the neighborhood of 20 feet per second were estimated.

The design so made was tested by an experimental model which was one-fiftieth actual size. The model discharged the calculated quantity of water, but showed that the water in the bellmouth reached a velocity that would not permit it to follow the curvature of the sides that had been laid out. The water piled up in places and formed stationary waves in the lower spillway that were larger than desired.

A second and third design were then made and tested by models. The results were interesting and satisfactory. They led to the adoption of a design with much less abrupt curvature in the bellmouth in which the flow was steady and well distributed.

The drop of water over the mica schist rock from the end of the spillway to the river bed, about 255 feet, remains to be considered. The rock in the line of discharge will be cleared of soil and debris for a certain width. It may be that flow of great quantities of water will loosen some of the rock and take it down to the stream especially as it is less solid at the surface than it is underneath.

If flows in 20,000 cubic feet were expected this would be a serious matter and the greatest precautions against the erosion of this rock would have to be taken. But flows of 20,000 cubic feet per second

are never expected. It is unlikely that any such quantities will ever come to the dam.

The maximum twenty-four-hour flood to be expected in a twenty-year period is about 3000 cubic feet per second. The reservoir may not be full when such a flood occurs. If it is, 1000 cubic feet per second may be drawn through the power house and as much more through the outlet gates in the diversion tunnel at the bottom if desired and most or all of the remainder may be absorbed in free-board storage. The spillway will need to function less than once in twenty years.

The spillway must be provided for remote contingencies and must be capable of taking large quantities, but, as a practical matter, it will not be used often, and the quantities passing it will not be large.

Erosion of the spillway would do damage to the channel at the power house and at the intake dam by filling them with fragments of the eroded rock. Extreme measures are not warranted in protecting the rock in the lower portion from erosion, that is to say, that part of it on the steep slope.

THE DAM SITE

The Cobble Mountain Dam Site was in a very wild bit of country and in a particularly inaccessible place. The Wildcat Road, a mountain road, used for logging and hauling lumber, was within a mile of it. This road was improved and a branch built to the engineer's office and to the dam site sufficient to take in equipment for preliminary operations. This is on the right bank of the stream.

Afterward the city built about 3 miles of new and substantial road from the Blanford Road, a state road north of the site, to a point on the left side of the stream near the top of the proposed dam. This was heavy construction, cut to easy grades and curves, largely in rock. This was done before the contracts for the dam and tunnels were let.

After the completion of the dam this road will be carried across the dam, around Cobble Mountain, over the spillway on a bridge and will pass the main gate house and extend to the Granville road which is the principal road leading south. It will also connect with the old Wildcat Road which will be the short route to other parts of the waterworks system and to Springfield.

Some other road changes and improvements will be required in connection with the reservoir but the city has been buying land to

protect the quality of the water during the twenty-four years since this source was first selected. The area now owned is so large that some of the local roads to be flooded may be abandoned and do not require to be replaced.

The estimated cost of the Cobble Mountain Dam and Reservoir and of the Power House and of all that goes with it, but not including the Borden Brook Reservoir built in 1909 nor the land, most of which had been previously purchased, nor other parts of the water supply system, is $6,143,000.

Contracts have been let covering the major parts of the work and a substantial part of the work has been done. Some things remain to be arranged. There is every indication at present that the work will be completed for a sum materially less than the estimated cost.

WATER METER PRACTICE[1]

By R. C. Warkman[2]

In purchasing water meters, it is good business to purchase a good meter, for water meters, like any other product, may be found to be inferior if we are not careful in their selection. First cost is not a true index of annual cost.

After purchasing a good meter, it is essential to take good care of it and handle it in the proper manner.

All meter manufacturers give certain instructions and warnings to be used when meters are being installed. These instructions should be carefully followed, particularly as to the rule to flush out the service pipe before setting the meter. Merely turning the stop cock on and off quickly does not suffice, as at times some foreign matter may have entered the service pipe at the main and is not quickly dislodged. Therefore, give the service pipe a good flushing until you are sure it is perfectly clean. Many a meter disc has been broken by failure to do this, and a great deal of revenue lost through the meter being stopped when first set. A strict rule to apply is that meter setters should never feel their job is complete, without assuring themselves that the meter is operating properly after setting.

Wherever possible, meters should be set outdoors, at or near the curb line, so that they are easily accessible to the meter readers. In addition, they are not as likely to be tampered with as basement meters are and there is less likelihood of unmetered branch line connections being installed. Meters should be uniformly set in a large enough meter box to permit easy removal or changes, and should be kept out of driveways. Many a meter box and service have been broken by not following this last precaution. The surest way to keep a meter out of a driveway is to set it inside the house lines. We may not know the location of the driveway when service line and

[1] Presented before the Kentucky-Tennessee Section meeting, January 26, 1929.

[2] City Water Company, Chattanooga, Tenn.

meter are installed, but we do know where the house line is, as, invariably, the house is set on the center of the lot.

The worst enemy of the meter is frost. More meters are damaged by this cause than by any others. Therefore, if your locality is subject to severely cold weather, even only at rare times, it is your duty to set your meter and service deep enough to take care of this condition. For outdoor settings the top of the meter should be, at least, 6 inches below ground level, in an air tight box. If meter box or its cover permits cold air to enter the meter housing, you are likely to have a frozen meter. For basement meters, a box built around the meters, and filled with sawdust, makes a good and inexpensive frost protection, where there is no heat in the basement.

New meters should be tested before installation, and a record of the test filed. If this is not done, you have no argument to offer, if your consumer claims the meter does not register properly. As a matter of fact, you do not know whether it does or not.

The testing of meters at stated intervals is perhaps the most important factor in the care of meters. It is the meter that is responsible for producing the revenue which must keep the water works going. Frankly speaking, this is a matter to which a great many of the water works officials give very little attention.

While it is hard to form a standard practice for all plants to follow, owing to the varying local conditions, at least some sensible method can be followed in the testing of meters. My idea (perhaps a great many of you will not agree with me) has always been that the proper time to test a meter is when it has delivered a certain amount of water, and not after it has been in service a certain length of time. This applies especially to the larger meters. If we stop for a moment to reason this out, we would admit there is no basis for testing two meters in the same length of time, when we know one of the two is delivering twice or possibly three times the amount of water as the other. If this rule is followed, stuck meters may be reduced to the minimum, except in case of frost, hot water or the entry of foreign matter into the meters from some source.

Small meters are easily removed from service, and they can better be tested in the meter shop. The large meters, however, are not so easily removed, and they may be easily tested in service, if properly set. For this purpose, all large meters should be set with a tee placed between an outlet valve, and the outlet side of meters, for the purpose of making a connection for a test meter. After such

test it is good practice to open the meter, regardless of how well it may test, and all parts showing wear should be renewed. Realizing it is generally impossible for the consumer to get along without water while the test is being made, it is good practice to install meters in battery in all large services, so that one meter can supply the premises while the other is being tested. If this is found impossible, a by-pass connection can be rigged up temporarily while test is being made. This by-pass could be metered temporarily or left unmetered, that being optional with the water works operators and dependent upon the time of making the test.

The question often arises as to what degree of serviceability may be expected from a meter. I believe, in all cases, the standards set down by the American Water Works Association should be followed, and old meters should be repaired so that they will conform to that standard. If a meter cannot be brought to such a standard, it would far better be in the junk pile than in service, as you may just as well have a clerk stealing money from the cash drawer as to have meters in service that do not register all the water passing through them. The meter is more the cash register of your plant, than the actual cash register in your office.

For the purpose of keeping a close check on your large meters, they should be read weekly by a competent man. In cases of unusually large consumers, it may be good practice to read them daily. In all cases where the consumption shows a large decrease from the average, the meter reader should immediately investigate the cause. If no cause can be found, the meter should be tested. In fairness to the consumer, if a large increase in consumption is found he should be notified of this fact so that he may locate leakage, if any. This practice results in pleasant relations between the consumers and the water works officials.

Using the proper meter for the work it is expected to do is also highly important. There are several types of meters, such as the disc, piston, velocity, compound, etc. Each was designed for certain uses and it would be well when you are in doubt as to what type of meter to use on a particular service, to consult one of the meter company representatives. They are usually well informed and anxious to help out.

Most important of all along these lines is never to overwork a meter. Next to frost and hot water, overworking a meter is the surest way known to ruin it and cause it to under-register. Great

care must be used in selecting the proper size meter to do the work expected from it. You would not think of assigning to a boy a man's duty. Therefore, do not expect a 2-inch meter to do the work of a 3-inch and to stand up under the strain.

If a water meter could talk it would say something along these lines, "Wash my face once in awhile, clean me out when I get clogged. My vital organs (the disc and gear train) get so worn and weak from constantly being on the go, they need renewing sometime. If you do this I'll show you lots of pep, and if you take care of me, I'll take good care of you."

ACCOUNTING AND FINANCING FOR A CITY'S UTILITIES[1]

By A. H. Strickland[2]

In treating the subject assigned to me for this occasion I shall necessarily refer to the systems, practices and experiences of the municipal plant in Kansas City, Kansas. Kansas City owns and operates its water and light utilities combined in one plant.

The water works was purchased by the city from the Metropolitan Water Company in 1909 at a cost of $1,097,850. In financing the original purchase of the property, bonds were issued by the city and since that time bonds have been issued in the sum of $2,914,600 for extensions and improvements. Bonds in the sum of $349,600 have been retired and the outstanding indebtedness on the plant on December 31, 1927 was $3,362,000. The problem of financing major improvements in our utilities under the Kansas law is rather simple as to process, but more or less embarassing and uncertain as to results, as the foundation of all such financing rests on the popular vote of our electors. This feature places the management of the plant at the mercy of the whims of the voters and of the attacks of those individuals who may be opposed to municipal ownership or to the machinations of politicians who may seek to embarrass the officials in charge. As far as the Kansas City plant is concerned, however, it may be said that the people of this city have always given the water plant generous support and its bonds have always carried by large majorities.

When the Department engineers have made estimates of proposed extensions and improvements to the property, the Board of City Commissioners authorizes the mayor to issue his proclamation calling an election and the proposition of issuing the necessary bonds is submitted to the voters. If the result is favorable, the Department is then in a position to proceed. Proceedings of the Board of City

[1] Presented before the Missouri Valley Section meeting, October, 3, 1928.
[2] Commissioner of Department of Water, Light and Power, Kansas City, Kans.

Commissioners are formed into a bond transcript which, together with specimen bond and financial statement, is submitted to nationally known bond attorneys and the sale of the bonds made subject to the approving opinion of such lawyers. When the bonds are actually sold and the proceeds of such sale turned over to the Treasurer, the money is placed in the Extension Fund of the Department. This particular fund under the rules of sound operation and good business is used exclusively for new work and is never diverted to operating or maintenance uses.

In addition to financing the major extensions as above set out, this plant has continually made improvements and minor extensions (which properly could have come from bond money) from the earnings of the Department. In fact, up to August 1, 1928, the sum of $727,043 was used in that manner and in addition to this $100,000 was recently transferred from the surplus earnings into the Extension Fund.

The State law under which this utility operates requires that such rates be fixed as will provide sufficient funds to cover all costs of operation, maintenance, bond interest, sinking fund and depreciation charges. In carrying out the intent and purpose of this Act, the management of the property has gone even further than the statute contemplates. The plant has been operated successfully, it has been maintained as nearly 100 per cent efficient as is possible, interest on the bonds has always been met and all bonds have been paid at their maturity. A sinking fund has been established on a scientific basis that will adequately meet all future maturities and, in addition, depreciation is being written off on a rate of 2.72 per cent. Over and above that necessary outlay the plant has been able to take care of many of its improvements through earnings as before outlined.

METER READING AND BILLING

The Department derives its ordinary revenue from the sale of water to its consumers, the rental of fire hydrants to the city government and from miscellaneous sources such as sales of meters, plumbers' permits, etc. In order that the Department might realize quickly on the sale of water, the continuous system of billing was installed. The Department has at this time approximately 27,000 water accounts which have been divided into 10 districts consisting of 192 routes. These routes are read continuously. Although meters are read continuously throughout the month the bills are so divided and

mailed out that it has been possible to establish ten days throughout each month upon which the penalites for non-payment of bills fall due. These accounts are handled by automatic machines of the Burroughs type made especially to handle the style of bills used by the Department. In addition to the 27,000 regular accounts, there are 500 commercial or large consumers which require individual attention and these bills are made by hand.

While the meters are being read, the addressograph department prepares a supply of bills for each district being read and upon completion of the reading, the billing is computed directly on the meter reading sheet. In addition to addressing of all bills, the addressograph department is also required to make a blank ledger sheet and meter reading sheet on all consumers. These sheets are made direct from the addressograph plate. They are then forwarded to the ledger clerk who makes corresponding entries on the ledger and meter sheets which are placed in their respective routes and the old consumers ledger sheets are transferred to the inactive file or the closed account file, depending upon whether the consumer leaves an unpaid balance or not. In cases of unpaid balances the charge is transferred to a history card file, arranged alphabetically by consumers names and set up against the consumer which is collectible on the present service or in case of non-service, the balance must be paid before new service is given. All orders of consumers applying for service are cleared through this file. The meter reading books having been billed are now ready for the billing machines, but they are first cleared by the control clerk who balances each route monthly, checking all outstanding balances in the controls with the report from posters showing cash posted in each route, which detects any errors in posting. This method proves the correct balances of the ledgers for each route and the control clerk is able to spread the balancing of the ledgers over the entire month. As soon as the last route is billed, final figures for the month can then immediately be made up and they are then transferred to the general control ledger to be checked by the accounting department. With the special machines now in use we can get the bills, ledger sheets and recapitulation of sales in one operation and are able to present the consumer with a neat, clean bill. By one operation we prove correctness of each bill before sending it out. If, by chance, the wrong keys on the machine should be used by the operator, the bills, ledger sheets and recapitulation sheets are adjusted by the billing clerk as soon as this particular route

is billed on the machine. The machine records the following: the prior reading, the present reading, the consumption and the total amount of the bill. The proof is as follows: by subtracting the total old reading from the total new reading, the difference will prove the consumption billed. Distribution is then made from the recapitulation sheet as to the rates. This sheet is used for the monthly report for water sold and finally for the yearly report as to how the water was distributed according to the rates.

The Department has at present three regular clerks operating the billing machines and is equipped with four machines. This gives us a surplus machine to be used in case of repairs on one of the regular machines. These machines handle approximately 95 per cent of all the accounts carried under the present system. The balance of the accounts are of such a nature that the bills must be made by hand. These regular operators average 800 accounts daily This number includes, of course, a proportion of electric light bills that are gotten out by the same operator. Each district has a printed penalty date on the bills which makes it mandatory that the bills be prepared on time in order to allow each customer ten days in which to pay the bill after it is mailed and before the penalty is applied. This schedule of penalty dates for each district has been kept in force unchanged during the last six months, except in such cases as fall on Sundays or holidays. The billing system now used in the Commerical Department of this plant has proven highly satisfactory because it has given regular periods between the bills and also enables the Department to have the regular penalty dates, but it has the greater advantage of getting the bills out with regularity thereby causing a better feeling on the part of the consumers. It also results in the Department collecting the revenue quickly after the water has been used.

In connection with this system it would only be fair to say that the Department had the benefit of the experience and advice of Price, Waterhouse and Company, who planned this system to meet the conditions as they exist in this Department.

COLLECTIONS

Collections are derived from three sources as follows: first, direct payment at the cashier's window; second, mail collections, third, through the different banks of the city. In the division of these collections we are able to ascertain any error occasioned by any one of the three sources. The record of the collections shows each individ-

ual account number and amount paid. A collection sheet is made daily of all transactions and when completed they are passed to the Billing Department together with the stubs, which, after they are checked, are used for posting purposes. Under the existing form of government in Kansas City the City Treasurer is custodian of all city funds. The Water and Light Department cashier makes up his report and sends it to the City Treasurer in triplicate. The original receipt is approved by the City Treasurer and City Auditor and returned to the cashier for future reference. At the end of each month a check is made between the water and light department and the City Auditor. In this way balances between the two departments are continually maintained. Receipted stubs for cash posting along with collection sheets are received daily from the cashier and set up in numerical order showing the total collected for each route. These stubs are then checked against the collection sheet and posted in their respective routes. After each route is posted, stubs are checked against the ledger posting which verifies cash collected for each route. Individual route collections are then transferred daily to the route control ledger where the cumulative collections for each day are used in balancing at the close of each month, to be checked by the Accounting Department. Routes are audited as penalty dates expire. Unpaid balances are made in delinquent form and a delinquent notice is mailed to the consumer giving the amount of the delinquency and notifying the consumer that, if the account is still unpaid after five days, the service will be discontinued. The commercial office checks all stubs and miscellaneous receipt slips against the cash blotter furnished by the cashier. At the end of the month, the Accounting Department accounts for all the miscellaneous receipt slips that have been issued during the month. The Accounting Department posts from the cash blotter daily to the cash book giving each account its proper credit. At the end of the month cash balances of the different funds are checked with the City Auditor who in turn has checked the daily collections which are turned over to the City Treasurer by the Water and Light Department cashier. The Accounting Department checks the credits to the various accounts with the cash book kept by the cashier. At the end of each month the Commercial office furnishes the Accounting Department with the monthly consumers billing. This billing is charged to Accounts Receivable Consumers and the various income accounts are credited.

The Commercial Department is compelled to furnish the Account-

ing Department with a separate adjustment for each error that has been made when a customer has been over-billed. There are, of course, other adjustments that have to be made on account of errors, such as duplicate and erroneous collections. When all cash collections are taken from the cash books, all billing and all adjustments have been posted to the general ledger and the general ledger has been balanced, the balance of the Accounts Receivable Consumers account is checked with the control of consumers billing and this account and control must balance.

WORK ORDER SYSTEM

The Work Order system is used to record the cost of all new work, all miscellaneous work, all maintenance and operating expenses on automobiles and all maintenance on equipment when such maintenance on one job exceeds $200, with the exception of maintenance on boilers and stokers where the cost of the job must exceed $1,000 before a work order is necessary.

The procedure of issuing a work order is as follows. The general foreman or person in charge requesting that work be done furnishes the superintendent with a written request giving an itemized estimate of the cost of the proposed job. This request is either approved or disapproved by the superintendent. If approved, the request is signed and given to the Accounting Department. An authorization and a work order covering the proposed work are then written. The authorization and work order are signed by the superintendent and the authorization is signed by the commissioner. The work order and authorization are made in five copies. The original of the work order and authorization are used in the Accounting Department as posting records, two copies are given to the foreman, one copy is for the superintendent's file and the other copy is for the office file. Upon receipt of his copies, the foreman is then permitted to start the work and all labor and materials are charged to the work order number which has been issued. The Accounting Department posts this cost daily to the work order. Upon completion of the job the foreman returns one copy of the work order showing the date that the work was started and the date completed. The Accounting Department checks the charges to the work order with the estimate and any appreciable difference is checked with the foreman. After all differences have been accounted for, the work order is cleared with a journal voucher, the proper extension, miscellaneous,

maintenance or operation account being charged and the work order credited with the amount accumulated on the work order. On the back of four copies of the authorization the difference between the amount estimated and the cost of the job is shown. One copy is given to the Commissioner, one to the superintendent, one to the foreman and the other is for the office file. A removal work order is issued for each withdrawal of equipment from service. This work order is charged with the cost of removing the equipment and credited with the salvage value of the equipment. The work order is then charged with the original cost of the equipment which has been removed and the proper extension account is credited. The work order is then cleared, allowance for depreciation being charged and the removal work order being credited.

It might be well to mention that it has been the practice of the Department to charge items to expense rather than new work where there is any question as to whether the cost of a job should be capitalized. It has especially been the practice to charge the cost of small betterments at the plant to expense. Although the cost of each job taken by itself is inconsequential, there is no doubt but that the aggregate over a period of years is a substantial amount.

RATE MAKING FOR WATER WORKS[1]

By James Sheahan[2]

The making of a rate schedule for any public utility has during the past few years developed into a scientific proposition. This has been brought about to a large extent by the fact that privately owned utilities in nearly all the states are now under the jurisdiction of public utility commissions and after many hearings before such commissions in connection with various utilities a fairly good standard of building a rate structure has been developed.

It makes little difference whether the utility be for water, electric or gas service, the same general principles of rate making apply. Inasmuch as this group of men is primarily interested in water works we shall refer in our discussion only to this utility.

PRIOR INFORMATION NECESSARY

The purpose of any rate schedule is to obtain revenues, so the first question to be answered is how much revenue is required. This, of course, is a problem for each individual utility, but all too frequently some of the major needs for revenue are entirely overlooked.

We all know, of course, that operating expenses must be paid including fuel and labor, oil and waste, repair bills of various and sundry kinds, and the cost of maintaining the property in good operating condition.

Further, if the property is privately owned, taxes of various kinds have to be paid, and bond interest, and dividends to stock holders.

On the other hand, if the property is municipally owned, free water is usually provided for fire purposes, street cleaning, schools and public institutions. In either case, whether the property be municipally or privately owned, there is a continuous depreciation of the physical property, either through actual wearing out of the component parts of the property, or due to inadequacy and obsoles-

[1] Presented before the Kentucky-Tennessee Section meeting, January 25, 1929.

[2] Superintendent, Water Department, Memphis, Tenn.

cence of equipment, so this most important item of depreciation should be taken into consideration in arriving at the amount of necessary income.

Another item to be considered is injuries and damages. This again, is a matter for individual consideration. The injuries and damages on one property might be almost negligible, while on another property of apparently the same character, the item might run into quite a sum of money. In any event, provision should be made to have in reserve at all times a reasonable sum of money to take care of injuries and damages, should they occur.

Another item that has now many advocates is that of carrying blanket insurance on all employed, in amounts varying from $500 to $2,000, depending on length of service. We believe it will only be a matter of a short time until this system is adopted by all public service companies. It will not be many years also until some form of pension system is developed by utility owners so that all employees who have spent a long and useful life in the service of the public may look forward to spending the twilight of life free from the harassing cares of poverty.

The owners of every utility should have vision of the future and provide in the prospective income, not only sufficient for immediate needs, but have in mind the growth of the business and the need of increased income with increased growth. Normally, of course, income is expected to increase with the expansion of the plant, but it frequently happens that we are called on to make excessive investment in new territory. It may take several years for the business in this new territory to produce sufficient revenue to pay any return on this investment. All of these factors should be given consideration in determining the revenues required for the proper handling of the property. A state public utility commission would, in its language, "fix the value of the property for rate making purposes." Regardless of whether the property be municipally or privately owned, this basic principle should apply: "What is the value for rate making purposes."

You should get sufficient revenue from your property to pay all operating expenses, including taxes, to take care of injuries and damages and depreciation requirements, and, if you are wise, you will also get enough to take care of blanket insurance on your employees. The depreciation requirement will be about 1½ per cent of the total property valuation.

In addition to all the items named, you should get enough to pay a return of from 6 to 8 per cent on the value of the property. It makes no difference who owns the property. If it does not earn a return on the investment it is an operating failure, as there is no justification for a municipality operating any public utility and paying for any part of the operation out of general taxation. The users of water should pay all the costs of the operation.

It may seem to you that we have gone far afield in doing so much talking, and up to this time say nothing about a rate structure, but, as stated before, it is necessary to know the income required before any rate structure can be developed. We have drawn your attention to some of the vital needs in the way of income so that you may start from a proper basis.

Coming to the rate structure itself, you are all probably connected with old established properties and you have good ideas as to the amount of water pumped daily, weekly and monthly. You also have certain rates at present in existence, and you know what revenue they produce.

The next question is, have you provided for all the items we have discussed before? If not, figure what the ones you have overlooked amount to, so that your new rate structure may take them into consideration. Next, go over your old rates and see whether or not they are in any way discriminatory.

We take it for granted, of course, that your systems are all fully metered. If they are not, they should be. Discrimination means the selling of water to different consumers of the same class at different prices. There is no excuse whatever for this, and you operators will find that a one price system to all alike is the ideal system to use. In fact, for a water system we see no need of more than one rate schedule, so designed that the large user naturally gets a lower rate per 1,000 gallons or cubic feet, or whatever your unit of measurement may be, than a smaller user.

Theoretically, I suppose, rates could be worked out that would make each class of consumers pay a theoretically correct rate, but in this business, like all other transactions of life, we must continually make compromises.

In the establishment of a rate structure we must compromise between theory and expediency. For instance, it might easily be proven that any customer paying only a dollar per month for water was not paying the expense of carrying him as a customer, let alone

paying any profit on his service. On the other hand, one dollar per month to a laboring man with small income is quite a tax. Yet as a human being he is entitled for himself and his family to reasonable consideration. It is your duty to lighten his expenses to the greatest possible extent and let his richer brothers take care of the deficiency.

We think, however, that a minimum bill of one dollar per month is the smallest water bill that should be rendered, and for this minimum bill a sufficient amount of water should be furnished to satisfy the moderate needs of the poorest customers. Of course, different rate schedules would have to be developed for different communities. In fact, there are many small properties where the minimum bill would have to be at least $1.50 per month to get the required revenue for the property.

We know of one property where the minimum is $1.50 per month and 80 per cent of all customers are billed a minimum bill each month, so that your exact minimum would have to be determined for your particular case.

The next thing to consider is the quantity of water covered by each step of your rates. In other words, if you start off with an initial step of 40 cents per 1,000 gallons for the first 5,000 gallons, this would cover 2,500 gallons for the one dollar minimum, and it would also cover probably 60 per cent of all your customers, as 5,000 gallons of water per month will satisfy the needs of almost any ordinary family of five or six people and allow them to bathe as often as they wish.

The next step might be 5,000 gallons for 35 cents per 1,000, and so on by steps down to your low rates for large consumers. Bear in mind, however, that your rates should be cumulative, that is, all customers should pay the same for the first block, and those using amounts in the second block should still pay the full price for the first block or step, plus the lower price for the second step, and so on for each of the other steps. The rate schedule should read, say,

40 cents per 1,000 for the first 5,000 gallons
35 cents per 1,000 for the next 5,000 gallons
30 cents per 1,000 for the next 10,000 gallons
etc.

Such a schedule is absolutely non-discriminatory. The little customer pays at the price shown for his block and the big customer pays the same price for that block plus the prices set out for each additional block.

After establishing your first step of the schedule, your next operation in designing the rate structure would be to determine by actual count of your customers the number falling in the first block and the income to be derived from them. Then take the second block and do likewise, and so on for each block.

In the actual practice you would probably set up certain tentative blocks and check your income from the various customers as applied to these tentative blocks. If you found the income so derived would be in excess of your needs, you might lower the rate for each block a certain percentage and so arrive at the desired result as to income, or you might lower the rates on the high blocks only, if the low rate block seemed already low enough, or you could, to get any desired result, raise the block rates at the high rate end and lower them at the low rate end of the schedule, or lower them at the high rate end and raise them at the low rate end.

We know of certain cases where several schedules of rates are used by the same company in selling water, say, one for domestic, one for commercial and one for industrial service. In Memphis we have one schedule only and whoever buys water within the city limits buys on that schedule. Customers outside of the city limits pay the schedules plus 50 per cent.

In conclusion, we suggest that, before attempting to make up a new rate schedule, you become fully conversant with what your needs really are. You may be surprised to find that you are overlooking some of the essentials to successful operation.

In our Memphis property we are always looking to the future and trying to anticipate the requirements from five to ten years hence, so that the dear public whom we are trying to serve, may never be able to say of us: "As public servants you have been weighed in the balance and found wanting."

FIRE PROTECTION SERVICE AND CONCENTRATION OF PROPERTY VALUES[1]

By H. H. Botten[2]

Concentration of property values is a natural result of the enlarged industrial activities of the present day and manifests itself in congestion in high value districts in cities and in a parallel condition in individual industrial plants. The resultant extraordinary requirements in the way of transportation facilities and various forms of public utilities such as power, lighting, heating, and domestic water and fire protection are apparent. These developments do not occur from a single, sudden expansion, as a rule, but from gradual accumulations and enlargements which sometimes outgrow the capacity of the public utilities serving them without attracting any special notice.

In your own city or nearby you have examples of this growth regardless of what industry may be the principal activity in your locality. Many of our plants which twenty years ago were considered large are completely dwarfed by the same plants of today. The importance of manufacturing and business enterprises viewed from the standpoint of payroll and general desirability to the communities in or near which they are located is usually in proportion to their size. It is a problem of special interest to the community to preserve such enterprises from destruction by fire. You are in the business of selling water and incidentally furnishing, usually without direct revenue, the water facilities for fire protection purposes. The destruction of a large manufacturing plant often means the loss to you of a large and profitable account and in addition to this there is usually a loss of many small ones dependent directly or indirectly upon the industry destroyed. A partial loss, if not too nearly total, does not discourage the owner from rebuilding, as it is generally advisable to do so rather than to abandon the standing portion of

[1] Presented before the Pacific Northwest Section meeting, November 16, 1928.

[2] Chief Engineer, Washington Surveying and Rating Bureau, Seattle, Wash.

the enterprise, but a total loss often results in the industry moving to a new or better location or in complete abandonment.

Economy of quantity production furnishes the principal reason for the huge increase in the size and value of the individual enterprises and in the face of the obvious advantages of large scale production, it is becoming increasingly unattractive for the small enterprise to go forth in competition with the industrial giants already well established. Economy of large scale production can only be effected by creating a situation which produces the maximum output with the minimum of space, labor and handling of the product while in the process of manufacture and after being finished. This produces a natural tendency to congest the occupied space and to conduct the complete manufacturing process under one roof, if possible, with the least number of obstructing partitions, fire walls and clear spaces so that the raw material may conveniently come into the factory at one end and emerge from the other ready for the market. This has resulted in what may be called reckless accumulation of value subject to one fire.

Several factors besides economy have contributed to this development. The control of building construction with respect to areas of fire sections, type of construction, occupancy, and other features which contribute to the fire hazards is not usually exercised by the State. Municipal governments can only control these matters within certain well defined limits. Naturally the city governments do not wish to drive industry away by enforcing extra drastic requirements and usually there is a zone in every town or city where fire regulations are very liberal or perhaps lacking altogether. At any rate, the manufacturer has only to go outside of the city limits to be free from restrictive regulations.

Credits today are practically unlimited and it is not difficult for a going enterprise to obtain sufficient capital regardless of its size.

Insurance, which is the foundation of our credit system, is likewise obtainable in practically unlimited amounts if the enterprise is prosperous and well managed. It is proper that credits and insurance should be plentiful and neither one nor the other can reasonably be expected to place limitations on the size of the individual risks or otherwise exercise police powers which should properly be administered by the legally constituted authorities.

If you have been able to keep up adequately with the demands on the water systems in your respective jurisdictions, you are to be

congratulated. However, I happen to know that in some instances the growth of the industrial districts has outstripped your facilities to a point where something needs to be done about it, but I am very glad to say that in the more important manufacturing districts in our own state, the water departments are making drastic enlargements of their plants to meet the increased demand occasioned by the new conditions.

If large manufacturing enterprises cannot be rendered reasonably safe against destruction by fire and other elements—and fire is the principal element of destruction to be guarded against—insurance companies are not inclined to underwrite the fire liability and the investors likewise are likely to decline to provide the financial backing required. Fortunately, there are means of rendering property comparatively safe against destruction by fire, even in the extraordinary situations mentioned. The automatic sprinkler has probably contributed more than any other one thing towards furnishing the necessary safeguard in this respect and it is now universally recognized as a means by which the desired security can be obtained, especially when combined with efficient public fire departments and water facilities.

As a result of the experience with automatic sprinklers, it is taken for granted almost universally that plans for any large new enterprise will include a complete system of sprinklers covering the sections which are subject to fire. Not only that, but progressive cities through their ordinances require automatic sprinklers in many situations not only for fire protection of property but as measures of public safety. Building laws usually permit larger unbroken areas, greater heights and other special features in consideration of complete sprinkler protection.

While, as stated before, the enlargement of the single unit fire risk calls for extraordinary fire fighting facilities, there is a happy circumstance connected with it which, if taken advantage of, is a means of obtaining the desired results with reasonable fire flow requirements; namely, the amount of water required for the average fire in a sprinklered plant or building is only a small fraction of the amount required for controlling fires in the same situation through the usual fire department operations alone. If we review the tabulated results of sprinkler fires for the years 1897 to 1927 inclusive, we find that 91½ per cent of the fires reported are successfully handled by twenty sprinklers or less and the water consumption for such fires

is almost negligible. Unfortunately, it is necessary to provide for the other $8\frac{1}{2}$ per cent of the fires which involve a much larger number of sprinklers. Should we fail to control nearly all of this remaining $8\frac{1}{2}$ per cent of the fires, the losses in the huge values involved would soon upset our calculations beyond all hope of a favorable result. It is accordingly necessary to provide for a much greater fire flow than indicated by the first $91\frac{1}{2}$ per cent of the fires in sprinklered risks. In addition to the requirement for sprinklers, some additional fire flow must be provided for fire department operations which almost invariably are necessary to control fires involving the opening of a great many sprinklers. Unless the sprinkler equipment has failed entirely in its functions, however, the fire department operations required are only incidental and do not call for the fire flow which would be required for the same risk unsprinklered. If the sprinkler equipment has failed, then the situation necessarily reverts back to that of an unsprinklered risk and the supply mains to the sprinkler equipment are invariably shut off to conserve the water supplies for the purpose of confining the fire as nearly as possible to the building in which it originated.

It is obvious that sprinkler connections of adequate size must be provided if full use of the automatic sprinkler equipment is to be had. If adequate connections cannot be obtained from public mains, then it is necessary for the property owner to shoulder the entire burden and provide his own water supplies. The cost of such supplies and their maintenance throughout the life of the plant has deterred many a property owner from installing this form of protection. From the thirty year experience from 1897 to 1927, it has been clearly demonstrated that the presence of a large sprinkler connection from a public main to a private fire protection system is not a menace to the water works system, but actually furnishes protection against crippling the water supply through the excessive drafts of water required for fire department operations with the property unsprinklered. With the supervision given by water works and insurance inspection departments, no connections of this nature are made without accessible gate valves outside of the risk protected so that in the event of a sprinkler failure the waste of water through the broken connections can be controlled by the fire department.

The reluctance on the part of water works utilities to meet the demand for adequate sprinkler connections is largely a relic of the past. It has been the natural result of more or less well founded

fear that the draft through a large sprinkler connection would deplete the fire flow in the immediate locality, but the standardized requirements of insurance underwriters and the up-to-date supervision given by the water works department have now minimized this hazard to a point where it has well nigh disappeared.

If you are still adhering to narrow restrictions as to the size of sprinkler connections, it is earnestly recommended that you set about to revise your regulations in the light of the experience that has been developed with this form of protection and out of consideration of your own interests. It is not recommended that fire connections be installed without meters or other suitable safeguards against surreptitious use or careless waste of water, but only the equipment necessary to safeguard the interests of the water works should be required and the cost of this should be reduced to the lowest practicable minimum. The property owner with enough thrift and enterprise to install an expensive system of protection which not only protects his property but reduces the hazard to that of others, and at the same time reduces to a very small percentage the average requirements upon fire and water departments for protection of his property surely deserves every consideration.

It is not very long ago since the restriction in size and number of sprinkler connections could be offset by inducing the owner to install automatic fire pumps drawing from some inexhaustible body of water unfit for anything but fire protection purposes or processing. This, as you know, in this state at least, is a thing of the past and the fire insurance interests locally have given you the best possible coöperation in the observance of the new regulations governing cross connections. This should be thoughtfully considered as it indicates an additional reason for advocating ample sprinkler connections from public mains.

We can still install elevated gravity tanks filled with city water for the purpose of augmenting the public supply, but these are usually intended to serve as emergency supplies in the event of the public water being temporarily out of service. However, gravity tanks are often objectionable from the standpoint of appearance, they are expensive to install, and at best only furnish a supply for a limited period of time. In large industrial plants a high pressure water supply which can be depended upon to furnish the full required fire flow for an extended period of time, five hours being usually considered the minimum, is absolutely essential. We cannot expect a gravity tank installation to meet such a requirement.

The additional fact that city authorities frequently compel property owners to install sprinklers throughout their premises indicates an obligation on the part of the city to furnish adequate water supply when available.

It is fairly safe to predict that the future will see the development of more rather than less large single unit enterprises and that the building up of manufacturing sections will continue to add to congestion of values, with the result that water works superintendents will have the problem of providing adequate water for fire defense constantly before them.

THE HYDRAULIC GRADIENT IN WATER WORKS MAINTENANCE[1]

By Elmer G. Hooper[2]

The three phases of water works engineering, design, construction and maintenance, all present a multitude of problems for the engineer and superintendent to solve, many of them peculiar to the physical characteristics of the locality in which the system is or is proposed and many peculiar to the individuals served and serving. Of the three phases I believe maintenance presents the most intricate, certainly the most trying, problems of all. Designing and constructing a new system involve the consideration of practically new material, unhampered for the most part by restrictions imposed by existing, possibly poorly conceived, structures. Even the enlargement of an existing system permits reasonable latitude to the designer and constructor in the selection and use of new methods and new material. On the other hand the supervisor of maintenance has to take what he finds, good or bad, adequate or inadequate, make it serve a temperamental public with complete satisfaction and continue to serve or suffer the penalty of constant reproach and of eventual failure.

In the solution of many of the problems there are often a number of methods that may be used with success, but not all with the same degree of success, and it is worth while to dwell for a moment on this important fact. Practically every method, every device, has a limit to its efficient application and it may suffer by comparison, suffer irreparable damage to its reputation, when applied with only mediocre success under conditions not ideally suited for it. Let me illustrate.

The measurement of water quantities may be made in a number of ways, by weight, volume, velocity-area, impulse and chemical

[1] Presented before the New York Section meeting, May 3, 1929.

[2] Associate Professor of Hydraulic Engineering, New York University, New York, N. Y.

solution, and with a great number of devices, tank and scales, tanks, displacement meters, floats, current meters, pitometer, Venturi meter, weirs, salt velocity device, salt or chemical solution apparatus, etc. It is quite obvious that a weir cannot be used in a pipe under pressure nor can a float; it is also reasonably obvious that salt or chemicals introduced into drinking water may be objectionable. On the other hand, it may not be so clear that the pitometer and the Venturi meter each may have some virtues in common for water measurement in pipes under pressure and yet one suffer by comparison with the other when put in the place where that other is preëminent. Certainly the Venturi meter is not as easily or economically installed for temporary measurements as is the pitometer and I believe the pitometer is not as satisfactory an installation as the Venturi meter for a permanent station. Enough has been said in this vein to emphasize the point that most everything has its peculiar place and its use should be limited to that place if it is to give satisfaction.

In the light of what has been said we may take up the consideration of the subject of the paper, the "Hydraulic Gradient." Like the methods and devices for water measurement just described, it is a most useful and singularly successful means for solving certain kinds of problems that come up in maintenance, but it is necessary to recognize its limitations in order to retain full confidence in it. I shall try to demonstrate by example later on in this paper the type of problem that may be readily handled and show how in one or two cases the method had to be supplemented by other means to complete the solution. First, however, I will ask you to bear with me while I go into hydraulic theory a little and try to interpret it in a way that is useful to the practicing water works man. Some of you may be bored because I am telling you something you already know, but I am sure there are others who have forgotten enough of it to make it worth while to refresh your memories.

THE THEORY OF THE HYDRAULIC GRADIENT

The "Hydraulic Gradient" may be defined as a continuous line or connected series of lines following the horizontal meanderings of the pipe or conduit but at a distance above or below it equal to the *height* to which the water from it *would rise due to pressure* in the main, either positive or negative. In other words it represents the limit of height to which water from the main could be delivered at any point. A sloping gradient indicates flow; a sudden drop in a

sloping gradient at any point indicates concentrated loss of head, as at a partially closed gate, an obstruction, or a change in diameter of pipe section if the gradient has a different slope following the drop. A gradient changing suddenly from a steep slope to a less slope without change of pipe diameter usually means a large concentrated outflow at the point of gradient change, though it may mean a flow from a very old pipe into a new pipe of the same diameter. A horizontal gradient indicates no flow or very little; when a flat gradient drops suddenly at a point and resumes beyond at a lower level it indicates a disconnection, a gate down or a complete obstruction. In all these the point at which the gradient breaks is the important one. A gradient that slopes too steeply indicates too much flow for that size of pipe or too much deterioration in the pipe to permit that flow; in any case a gradient that is found to be much steeper than it normally is usually means excessive draft, probably leakage.

The mathematical expression for the hydraulic gradient may be obtained by transposing Bernoulli's Theorem, which must be familiar to all who have studied hydraulics in recent years. The theorem may be written in the following form between any two sections of the main

$$\frac{v_1^2}{2g} + \frac{p_1}{w} + z_1 + h = \frac{v_2^2}{2g} + \frac{p_2}{w} + z_2 + L.H.$$

where v represents velocity at the respective sections,

p represents gauge pressure if w is 0.434,

z represents elevation of the pipe at the respective sections,

h represents lift from pump if there is one between the sections,

$L.H.$ represents the sum of all losses of head between the sections,

and $\frac{v^2}{2g}$ is the height in feet to which the water would rise due to velocity alone; $\frac{p}{w}$ is the height to which the water would rise due to pressure alone if an open vertical tube were inserted in the pipe (*the height of the gradient* above the pipe at the point); z is the height of the section of pipe above a common datum; the losses of head are divided for convenience into those due to friction in pipe of uniform diameter, to bends, to expansions and to contractions of sections, and then summed up. If a common level or datum is used it may

be seen that the term $(\frac{p}{w} + z)$ is the height of the gradient at the section above that common level, and that $(\frac{p_1}{w} + z_1) - (\frac{p_2}{w} + z_2)$ is the *drop of the gradient* between the two sections. If we transpose Bernoulli's equation to produce this form we will have the following

$$\left(\frac{p_1}{w} + z_1\right) - \left(\frac{p_2}{w} + z_2\right) = \frac{v_2^2}{2g} - \frac{v_1^2}{2g} + L.H. - h$$

meaning that the gradient drops due to loss of head, rises due to lift of the pump, drops if the velocity increases and rises if the velocity decreases. For practical water works purposes outside the pumping station all items except loss of head may be neglected and we may say then that the drop in the gradient is from loss of head. The drop in the gradient is therefore a measure of the efficiency of the parts of the system, and the nature and magnitude of the drop may be interpreted to tell whether anything is wrong, and may indicate what and where.

APPLICATION OF THE GRADIENT TO PRACTICE

To illustrate the application of the gradient to practice, I am drawing on my memory of experiences in the service of the Department of Water Supply, Gas and Electricity, New York City, and because of the ten years or more that have elapsed since that service only a few outstanding examples are clear enough in my mind to present them. As the cases are described you will no doubt say to yourselves, "why, he is only taking pressures." I shall fully agree with you, for it is the pressure reading converted to feet head and added to the hydrant elevation that gives the gradient elevation. You will appreciate then that it is only a common, everyday job that can be put to very useful ends when properly interpreted.

Case 1. An abnormally steep gradient, indicating excessive flow on a 12-inch main in 43rd Street, Manhattan, between 5th and Madison Avenues. A casual pressure taken on a hydrant on the north side of 43rd Street east of Madison Avenue seemed to be much lower than the district should have and this was verified as fact by pressures taken on mains in the same service not closely connected to the one suspected. This latter main had no connections to any other mains at or near Madison Avenue and 43rd Street, though had

there been any they would have been closed temporarily to permit flow from the two ends only, for the purpose of the investigation. The problem then consisted in determining and analyzing the gradient on a line 12 inches in diameter beginning at a 20-inch trunk main in Fifth Avenue, extending east through 43rd Street across Madison Avenue, where it changes to 6 inches in diameter continuing east to Vanderbilt Avenue at Grand Central Terminal, then north one block to 44th Street in which it turned west to connect with a 12-inch main in Madison Avenue. There is a valve in 43rd Street east of Madison Avenue but west of the hydrant on which the first pressure was taken. This valve, a 12-inch, will be designated as valve 1 for future reference. Valve 2, a 6-inch, is on 44th Street east of Madison Avenue. There were five hydrants on the line at which pressures were taken and converted to feet head. Elevations of these hydrants were determined by hand level readings from the most convenient street intersection whose elevation was known or could be assumed for datum, and the spacing of the hydrants was measured by pacing. By adding the pressure head to the elevation for each hydrant and plotting it to scale for the whole line a gradient was found consisting of two sections, each having a fairly steep slope downward from the ends of the main toward a common point about on the east building line of Madison Avenue and 43rd Street, near valve 1. In order to find on which side of this valve the outflow occurred, it was closed so that the supply could come from only one direction. The hydrant pressure dropped to less than zero, showing that the flow was from the east through the 6-inch line to a point just east of the valve, and that the amount of outflow was equal to the capacity of that line at least. For checking the location of the outflow and to obtain data to compute the amount, valve 1 was opened and 2 was closed and simultaneous pressures were taken for a new gradient.

Investigation showed that Grand Central Terminal improvements extended under the block from Vanderbilt to Madison Avenue east building line; that the main dropped several feet to pass between the floor of the basement and the ceiling of the sub-basement; that a 6-inch blow-off had been installed and connected with the sewer at that level in the west wall of the basement; that a special operating wrench for the blow-off gate had had to be provided; and that finally this gate was wide open thereby filling the sewer, a large one, more than two-thirds full. It was estimated from the pipe size and loss of head that about 4 m.g.d. was wasted and the sewer marks indicated it had been doing so a long time.

Another example quite similar to the above, except that it was on a submarine line, was taken as the subject of a brief paper read before a meeting of the New England Water Works Association in 1916.

Case 2. Gradient slightly lower than normal on a trunk main where pressure variation was usually slight; pressure reading was only about a pound lower than normal on a hydrant from a 20-inch main a few hundred feet east of its connection with a 48-inch main. This was on Pelham Parkway east of Southern Boulevard, the Bronx, on a main leading about 6 miles to City Island, having two 12-inch and one 8-inch lines at intermediate points normally feeding out toward Westchester village. When the gate was closed on first 12-inch feeder the gradient showed that the flow had been into the 20-inch main; the same thing was indicated at the other two lines showing a heavy draft toward City Island. At two points the 20-inch line divided into two 12-inch lines for under water crossings and the closing of these in succession affected the gradient in such a way as to show the supposed trouble to be still farther ahead and the gradient was becoming noticeably lower and lower. The heavy flow was finally pinned down to an 8-inch under water line about a mile long, which together with a 12-inch line supplied Hart's Island with water. When this crossing was shut off the local pressures went up from about 25 to over 40 pounds. The slope of the gradient was used to determine the approximate location of the leak and by using a boat, which acted as a good aquaphone, the location was checked and buoyed for the repair crew. This leak or series of leaks (three lengths of split pipe not far from each other) was under 16 feet of water at low tide. Less than a day was consumed from the time the suspicion was first aroused until the trouble was located, and for the most part by hydraulic gradient.

Case 3. Flat gradient followed by sudden drop. On one of the early days of 1917 when troops were being assembled and trained at Fort Schuyler, N. Y., water was suddenly cut off from the post. After considerable delay and late in the day word of the trouble was received. From the story of those on the ground the easiest and most obvious thing to produce results seemed to be pressure readings on hydrants outside the reservation since it was known there was no pressure within. The first reading taken showed a pressure slightly *above* normal, indicating at once a plugging of the line. Further investigation discovered a 6-inch meter with a strainer just inside the reservation and a very large eel tightly squeezed into the

strainer thereby shutting off the entire supply. The trouble was located in about a half hour and remedied in another hour.

Another somewhat similar procedure discovered a rather unique plug in a section of 6-inch water line at 140th Street and East River which connects with an under water line to North Brother's Island. About 50 feet of fire hose without couplings and tightly folded into the pipe by unbalanced pressure, produced by its own resistance to movement, completely shut off the supply of water to the island from the Bronx mainland, leaving the island dependent on another submarine line to Riker's Island and the mainland beyond. This was discovered when investigating the adequacy of fire protection for the island. The reason for the hose being in the pipe was learned later, but it would take too long to tell it here.

Under this case also would come the examples of gates closed and of pipes cut and capped. Such conditions were met with a number of times and it was not possible in any of the cases to tell which of the two were responsible until an excavation and an examination had been made. Because this was not an unusual or startling condition no particular one stands out in mind sufficiently clearly to be reproduced.

In the preceding paragraphs enough of both theory and of practice has been presented to provide food for reflection and it is to be hoped that some new and workable ideas have been suggested to some of you who have had the patience and the kindness to listen.

DISCUSSION

Edgar K. Wilson:[3] Prof. Hooper has presented a paper which should be very helpful to water works maintenance. He has called attention to a very useful tool which can be used in many ways to obtain various results.

On my desk I have a little volume with the following title page: Calculus Made Easy, being a very simple introduction to those beautiful methods of reckoning which are generally called by the terrifying names of the *differential calculus* and the *integral calculus*. The book is by Sylvanus P. Thompson, F.R.S., and in his prologue he mades this statement: "The fools who write textbooks of advanced mathematics—and they are mostly clever fools—seldom take the trouble to show you how easy the easy calculations are. On the

[3] Chief Engineer, The Pitometer Company, New York, N. Y.

contrary, they seem to desire to impress you with their tremendous cleverness by going about it in the most difficult way."

Like many engineering and mathematical terms, the designation, "Hydraulic Gradient," sounds somewhat formidable, and Prof. Hooper has done much in his paper to simplify its meaning.

I suppose almost every water works operator has at some time taken the pressures on two hydrants, corrected them for difference in elevation and found the loss of head between them; or at least has taken the difference between two hydrants at about the same elevation. By this he has more or less roughly obtained the hydraulic gradient between the two points. This is about the simplest example and unless connecting lines are valved off so as to require the water to feed in only one direction, the results are not of much value; but it shows that we are using the principles on which the hydraulic gradient is based in every day practice.

When these pressures are reduced to feet and added to the elevation at the point under consideration referred to a common datum, they may be plotted to a suitable horizontal and vertical scale, and the line joining the plotted points gives the hydraulic gradient. As Prof. Hooper has explained, this is the height to which the water would rise in a vertical pipe at any given point under the given conditions of flow.

Since the gradient is governed largely by the friction losses in the pipes, it is obvious that varying velocities will cause the gradient to vary likewise; and we find that certain sections of a system may have plenty of water at night when the velocities are decreased; while with the full consumption throughout the system, very little or no water is delivered to these sections during the day. This is a common occurrence, and one with which some of you are probably familiar.

It is often possible by taking a series of pressure readings between the source of supply and these points of low pressure, to learn where new mains may be constructed, which may obviate the necessity of a booster system to give proper serivce. Obstructions may also be indicated, and the existing mains when cleared may be adequate without further expense.

A case which came to my attention a few months ago is a good illustration of this. A hydraulic gradient was constructed for a small water system several miles long from the reservoir to the lower end; and it was found that one point about half way was considerably lower on the grade line than the next succeeding point.

The principal feeds at this section were two parallel mains; and an investigation showed that no side connections were taken off the newer line. It was also found that the valve at the upper end of the old main was closed. As a result the water had to feed through the entire length of the new main and half way back through the old main before it could enter the distribution system; with a resulting loss of head of several feet. When the valve was opened the point in question took its proper place in the grade line, while the lower end of the system received about 5 pounds more pressure due to the increased feed.

It is evident from this example and those cited by Prof. Hooper that the principles of the hydraulic gradient may be very usefully applied to the solving of many water maintenance problems.

DRY SQUARE BRAIDED HEMP FOR YARNING JOINTS[1]

By Otto S. Reynolds[2]

The merits and economy in the use of dry square braided hemp are so evident at this time that many water works engineers and superintendents specify it on all bell and spigot work. There are several hemp manufacturers who carry it as a regular item. There are many water works supply houses throughout the country who also stock this material in 50- and 25-pound reels. The material is made up principally in three sizes. The $\frac{1}{2}$-inch square is the standard size, which is designed to fit all diameters of bell and spigot pipe from 4 to 16 inches inclusive. The annular clearance in the bells is designed to be uniform for the same class of strength and weight pipe. The $\frac{3}{8}$-inch square is furnished to meet the requirements when a spigot of fitting is entered into a pipe bell, the clearance being close. Likewise the $\frac{5}{8}$-inch square is furnished to meet the requirements on reverse conditions.

Braided hemp for the water works field should be purchased of a good quality. The standard $\frac{1}{2}$-inch size, free from any treating compound, should run about 16 feet to the pound, the undersize, $\frac{3}{8}$-inch, 25 feet to the pound, and the over size, $\frac{5}{8}$-inch, 10 feet to the pound. The material should be of a reasonably long fiber hemp. Like a rope, the longer and better grade of the fiber, the better the hemp. It should be strong to withstand the punishment of the yarning tool when being forced into the bell and not shear in two or break allowing an opening for the molten jointing material to run through into the inside of the pipe. This incident causes delay in the progress of the work. Besides it is an unpleasant thing to overcome. There is little difference in the cost of a short fiber hemp from a long fiber. Therefore, the longer fiber hemps are recommended.

[1] Prepared for a subcommittee of the American Railway Engineers Association, on revising railway specifications for laying cast iron pipe. Data based on practice in Indianapolis, Charleston, S. C., St. Louis, Fort Worth, and Independence.

[2] Traveling representative, The Leadite Company, Kansas City, Mo.

In the use of square braided hemp the hemp is cut to length allowing a lap of about 2 inches for 4- and 6-inch pipe; $2\frac{1}{2}$ inches for 8, 10 and 12 inches; and 3 inches for 16 and over. On account of the various makes of pipe, specific lengths to cut hemp would be numerous and difficult.

In application, a length of hemp is held on the extreme end of the spigot end of the pipe. The worker guides the spigot, holding it close to the top of the bell. At his word another man at the opposite end or leading bell end with a stick forces the entrance of the spigot into the bell, and the hemp rolls in on the bottom at the same instant. Be sure the pipe is "homed" well. Then with the yarning tool, without a hammer, push the hemp back to the bead or centering ring of the pipe on the bottom. This can be done easily, as the pipe is loose in the bell and care must be taken not to push the hemp over the bead or by the centering ring. Then with the assistance of a hammer, force the hemp in along both sides, and then on top letting the lapping end lead downward, that the molten material will not force an opening between the lap and on into the inside of the pipe. This completes the yarning. For lead, usually two raps around the pipe is taken, which leaves exactly 2 inches for the lead on standard pipe. For Leadite, but one strand is used, which leaves $2\frac{1}{2}$ inches depth for the Leadite.

This method of yarning does not require any special tools. No preliminary rolling or twisting is necessary. It does not require the skill of a mechanic to center it, pack it and space it for depth, as is required with the open jute. It will not burn through, as the material is very closely braided. A very nicely centered pipe is obtained, which is the success of any joint. A uniform depth is obtained for the jointing material, which effects a saving on the jointing material in that an exact amount of it is assured of being installed and little chance for an excess being used. As this method of yarning is very easily applied, greater speed is accomplished and there is very little chance of injury to occur. The pinching of the fingers, which often happens when centering up a pipe with wedges preparatory to installing open jute, is practically eliminated.

With Leadite as the jointing material, square braided hemp is highly recommended, because it is believed much better joints may be had, and, as only one-half the material is required compared with lead and open jute, it is without doubt more economical. Lastly, if a strand of open jute should be left extending outward into the bell

and then the Leadite be poured into the joint, it would remain there forming a conductor for a water leak, which would require cutting out and repairing. The Leadite is not hot enough to burn up the open jute, as would be the case with molten lead.

In braided hemp there is practically no waste to the material. The hemp is cut to exact length. It is not used for a rag to clean mud from tools, the hands or shoes. It is not used to wipe out the bells or spigots of the pipe. It is not used for starting fires, nor is it used to wad up and place into the end of the pipe for over-night at the end of the day—and sometimes overlooked to be taken out the following morning. It will not drive over and into the pipe to float about and possibly clog up a service or stick up a meter, both of which are expensive maintenance charges. Lastly, if a piece of hemp is dropped along the ditch, it is recovered, which is never the case with open jute, as the latter gets full of dirt and the yarner will not use it.

Many waterworks superintendents and engineers feel that, over the older method of yarning with open jute, dry square braided hemp is well worth its additional first cost, and that very good economy is effected in its use.

OPERATING EXPERIENCES AT THE SACRAMENTO FILTRATION PLANT[1]

By Ralph A. Stevenson[2]

The city of Sacramento, California, is situated on the Sacramento River immediately below the mouth of the American River, the famous stream on which gold was discovered in 1848. The city has always taken its water supply from the Sacramento River, although for about five months of the year, due to differences in flow of the two rivers, the supply actually comes from the American. This latter stream rises in the Sierra Nevada Mountains some 90 miles northeast of Sacramento. Its drainage area of about 2000 square miles is almost entirely mountainous in nature, with sparse population, and except for the spring runoff its water is clear and reasonably free from bacteriological and biological pollution.

The Sacramento River and its tributaries rise near the Oregon line, draining an area above Sacramento of some 23,000 square miles with a population of about 165,000. As these streams flow through the Sacramento Valley their waters are used extensively for irrigation of field crops. This irrigation has a most important bearing on the quality of the water supply, as a large part of it is used to flood rice fields. Within a distance of 80 miles of Sacramento there is planted yearly about 150,000 acres of this crop, the practice being to flood the fields to a depth of 6 inches and occasionally draining and reflooding. This is continued from May until September when the fields are drained for the harvest. If one can picture a lake 150,000 acres in area, 6 inches deep, standing in our brilliant California sunlight throughout the summer months, one can readily visualize the extent of algal growth present. The drainage from this section flows into the Sacramento River above our intake imparting a decided odor to the water, and this constitutes the major problem of purification with which we are confronted.

[1] Presented before the Water Purification Division, the San Francisco Convention, June 14, 1928.

[2] Superintendent of Filtration, Sacramento, Calif.

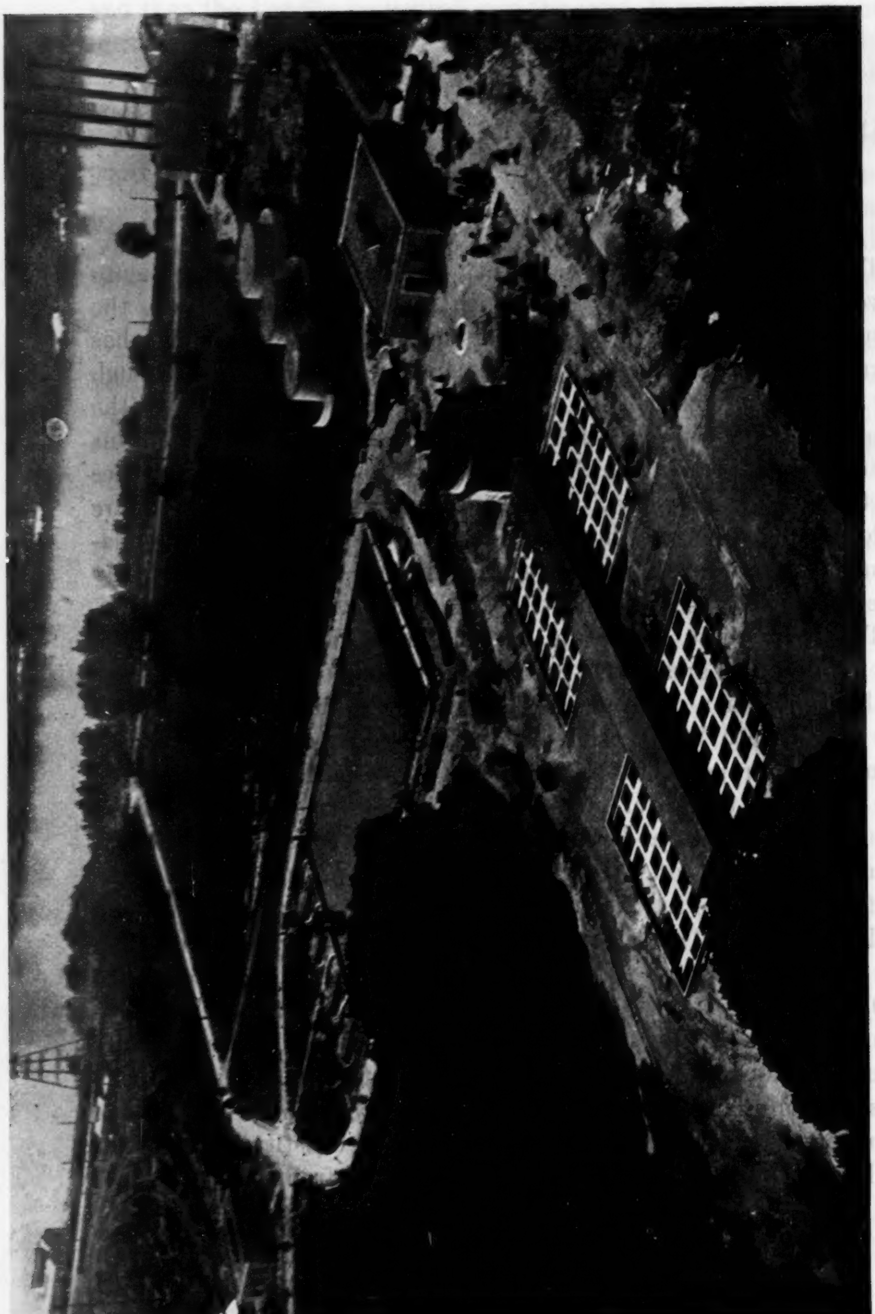


FIG. 1. AERIAL PHOTOGRAPH OF THE SACRAMENTO FILTRATION PLANT

Lower center, filters and head house. *Right center*, pumping station. *Upper center*, aerators, storage basin, (sedimentation basins under) and mixing tanks. *Upper left*, Hoover process alum plant. The Sacramento River in the background.

Construction was begun in January, 1921, on a modern rapid sand filtration plant to purify this water. Operation began in January, 1924.

A description of the plant begins with the intake pier, situated in the Sacramento River 1500 feet below the mouth of the American River. This structure, like the entire plant, is of reinforced concrete construction and is located about 60 feet from the shore line at low water. There are five gates at different elevations to allow water to be taken into the pier near the surface.

Two 60-inch conduits lead from the pier to the bank and over the river levee in the form of a siphon to the low lift side of the pumping station. This station has an installed capacity of 80 million gallons daily. From here the water is boosted to the aerators.

The aerators are 420 in number and placed at 4-feet centers, and are of the floating cone "Sacramento" type. The aerated water drops from 3 to 6 feet into a grit basin and flows across a storage basin of 1.5 million gallons capacity which is located on top of the sedimentation chambers. From the storage basin the flow is to the coagulant mixing tanks. These tanks, four in number, are round concrete structures 44 feet in diameter by 24 feet deep. Mixing is accomplished by means of paddles operated by reciprocating water engines mounted over the tanks. The total mixing time is about one and one-half hours.

From the mixing tanks the coagulated water flows through three sedimentation basins of 1, 2, and 3½ million gallons capacity respectively, with a total retention period at a 32-million gallon rate of about five hours.

The coagulated and settled water flows to sixteen filters of 4 million gallons capacity each. These filters are of the conventional rapid sand type with a sand surface of 1400 square feet, and having the Harrisburg distribution system. The rate of filtration is controlled by the height of water in the filtered water storage reservoir. This reservoir is covered and has a capacity of 5 million gallons. It is connected directly to the high lift pumps which operate against a closed system and maintain a pressure at the plant of 60 pounds per square inch.

The coagulant used is aluminum sulphate and is manufactured at the purification works by the Hoover process. This plant has a capacity of 3600 tons per year. The alum sirup is conveyed to the point of application, a maximum distance of 560 feet, by means of a

TABLE 1

Summary of chemical, bacteriological, and biological analyses of Sacramento water supply for 1925, 1926, 1927

TEST	AVERAGE		MAXIMUM		MINIMUM	
	River	Tap	River	Tap	River	Tap
	Averages					
	p.p.m.	*p.p.m.*	*p.p.m.*	*p.p.m.*	*p.p.m.*	*p.p.m.*
Chemical:						
Alkalinity	62	52	154	134	18	5
Chlorides	18	18	80	71	1	2
Hardness	62	61	176	168	12	18
Carbon dioxide	2.7	9.1	10	25.8	0	3.8
Oxygen consumed	2.8	0.8	27.7	5.3	0.8	0
Dissolved oxygen*	98	104	130	139	74	65
pH	7.3	6.9	8.0	7.2	7.0	6.8
Physical:						
Turbidity	47	0	650	0	15	0
Temperature (°C.)	15.5	16.3	27	28.5	3	5.5
Bacteriological:						
Total count per cubic centimeter	421	2.16	25800	50	40	0
B. coli index per cubic centimeter	6.3	0.0003	110	0.09	0	0
Biological:						
Algae† count per cubic centimeter	1200	0	18000	0	0	0

Mineral content, average 1927, Tap water, parts per million:

Dissol. solids	101.7
Silica	20.0
Sodium-Potassium	8.54
Sulfate	19.9
Iron	0.05
Aluminum	2.11
Carbonate	none
Calcium	10.67
Magnesium	4.29
Bicarbonate	53.4

Mud removed, by sedimentation basins—average year, cubic yards ... 4,356

TABLE 1—*Continued*

Alum dose, grains per gallon:	
Average	1.2
Maximum	5.5
Minimum	0.5
Chlorine dose, tap water:	
Average	0.2
Maximum	2.0
Minimum	0.15

* Per cent saturation.

† Predominant forms: Diatomaceae; asterionella, synedra. Cyanophyceae; anabaena, noctoc, lyngbya. Chlorophyceae; spirogyra, chaetophora.

Schutte-Koerting ejector operated by water pressure. The rate of dosing is controlled by the rate of filtration.

Chlorine is also manufactured at the plant by eight 600-ampere electrolytic cells designed and operated by the plant staff. These cells have a combined capacity of 300 pounds of chlorine per day. Control is effected by means of a rheostat which varies the amperage of the cells making it possible to manufacture the exact amount of chlorine required at any time.

We have within the month placed in operation eight new filters designed along the line of the original installation making the total filter capacity 64 million gallons daily at normal rate. Within a year we hope to have in operation an additional pre-treatment works designed for peak loads of 60 million gallons daily.

The operating staff consists of a superintendent, chief engineer, three operators and three oilers at the pumping station, one machinist and helper, an electrician, a chief operator and three operators at the coagulant plant, a chief operator and three operators at the filters, two relief operators, two chemists, three laborers and a janitor, a total of seventeen men.

CHARACTER OF WATER TREATED

The character of the water supply is shown in table 1. In general the river water is soft, reasonably clear, and except for the spring flood period, the B. coli index is low.

The location of the intake pier is such that the dividing line of the American and Sacramento River waters during the spring and early summer weaves back and forth across the intake, and as the two

rivers are decidedly different in character—the American being for the most part very soft and clear and the Sacramento being comparatively hard and turbid—it makes for flashy conditions. The alum dose has changed as much as three grains per gallon in the course of one hour and turbidities have jumped from 50 to 300 p.p.m. in a few minutes. From June to September the turbidity of the Sacramento River is about 30 p.p.m. due almost entirely to algae and other organic matter. The temperature during this period averages 23°C. and occasionally rises to 27°C. This high temperature intensifies the odor of the water and makes its removal difficult.

During this season the raw water is heavily chlorinated before aeration to oxidize the essential oils responsible for the odor. A dose of 1 p.p.m. is applied as the water leaves the pumping station. The contact period is about five minutes before aeration. This leaves about 0.2 p.p.m. of free chlorine for the aerators to remove which they do very efficiently. This treatment, because of the very short contact period possible, is not as effective as it might be, but it does remove a great percentage of the odor.

One effect of pre-chlorination is to cause aftergrowths in the sedimentation basins. About fifteen minutes after pre-chlorination and aeration the water is practically sterile, but in from five to six hours as the water leaves the sedimentation basins the bacterial count has at times jumped to 2000 per cubic centimeter, and at all times is higher than the water before chlorination. Fortunately, these bacteria are not of the type that give a spurious presumptive test so that no sanitary significance is attached to a high count in the applied water.

During the winter months when the water is very soft and the turbidity is high, a large alum dose is necessary to properly coagulate the water, and as the carbon dioxide evolved is considerable, red water complaints are likely to be numerous. In an effort to improve this condition coagulation before aeration has been successfully demonstrated, and as the water immediately after aeration is rather quiescent, only a small amount of very fine floc is formed so that it is literally possible to store the alum in the water as it flows across the storage basin and build up a good floc with it when it arrives at the mixing tanks.

It has been found that when the alum dose exceeds two grains per gallon successful coagulation is not obtained with one dose. When this condition prevails it is the custom to apply about three-fourths

of the total dose at the inlet of the coagulant mixing tanks, and the remainder at the outlet of the first sedimentation basin. This split treatment has always resulted in acceptable applied water.

In the late summer when most of the turbidity is of an organic nature it is possible to build up a large floc in the mixing tanks, but of such a friable nature that it is broken up between the mixing tanks and the sedimentation basin and does not again form. A plant scale experiment of adding mud to the water, before the alum, was tried and proved successful. As little as 10 p.p.m. of added turbidity

Fig. 2. Sacramento Floating Cone Type Aerators in Action (Operating at About 25 m.g.d. Rate)

strengthened the floc so that it would not break up, but because of the amount of dirt involved the plan was not considered practical, and instead during this period the practice is to by-pass the mixing tanks and apply the alum at the entrance of the first sedimentation basin which results in a fair applied water.

FILTRATION

The filter underdrains are of the Harrisburg type with one layer of 3-inch cobbles around the laterals supporting 6 inches of $2\frac{1}{2}$ to $1\frac{1}{2}$, 4 inches $1\frac{1}{2}$ to $\frac{3}{4}$, 4 inches of $\frac{3}{4}$ to $\frac{1}{4}$, 6 inches of $\frac{1}{4}$ to $\frac{1}{10}$ gravel and

from 24 to 28 inches of sand effective size 0.40, uniformity coefficient 1.28. The distance from the sand surface to the tops of the gutters is 24 inches.

Considerable trouble has been experienced from sand dropping down through the gravel and blocking the laterals. Several things

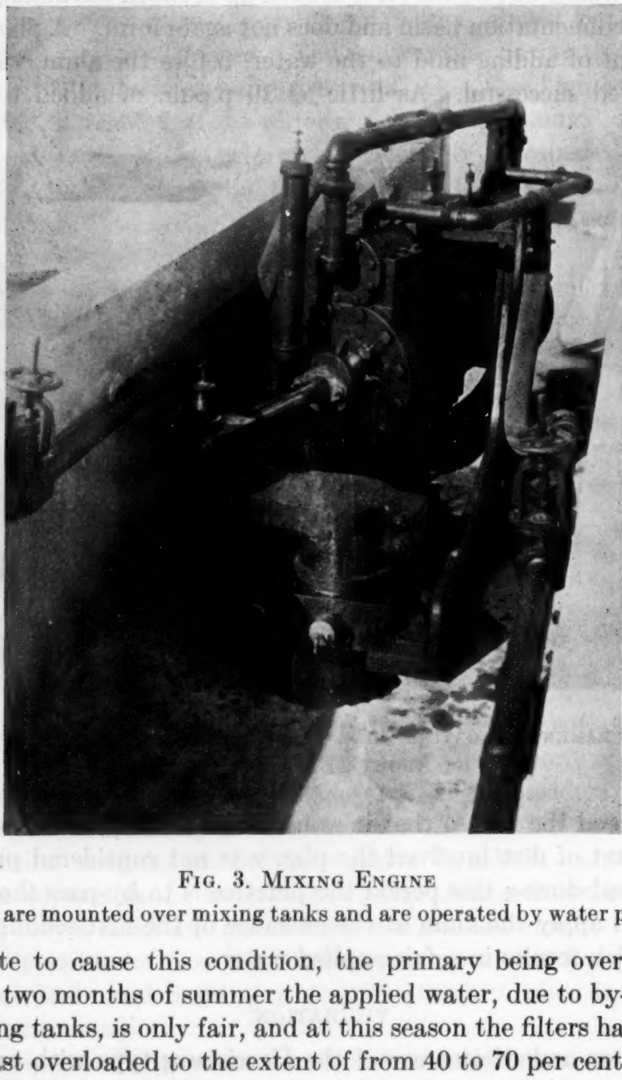


FIG. 3. MIXING ENGINE
Engines are mounted over mixing tanks and are operated by water pressure

contribute to cause this condition, the primary being overloading. The last two months of summer the applied water, due to by-passing the mixing tanks, is only fair, and at this season the filters have been in the past overloaded to the extent of from 40 to 70 per cent. Mud balls accumulate, rapidly cutting down the effective sand bed area and, unless pre-chlorination is practiced, a heavy mat of living

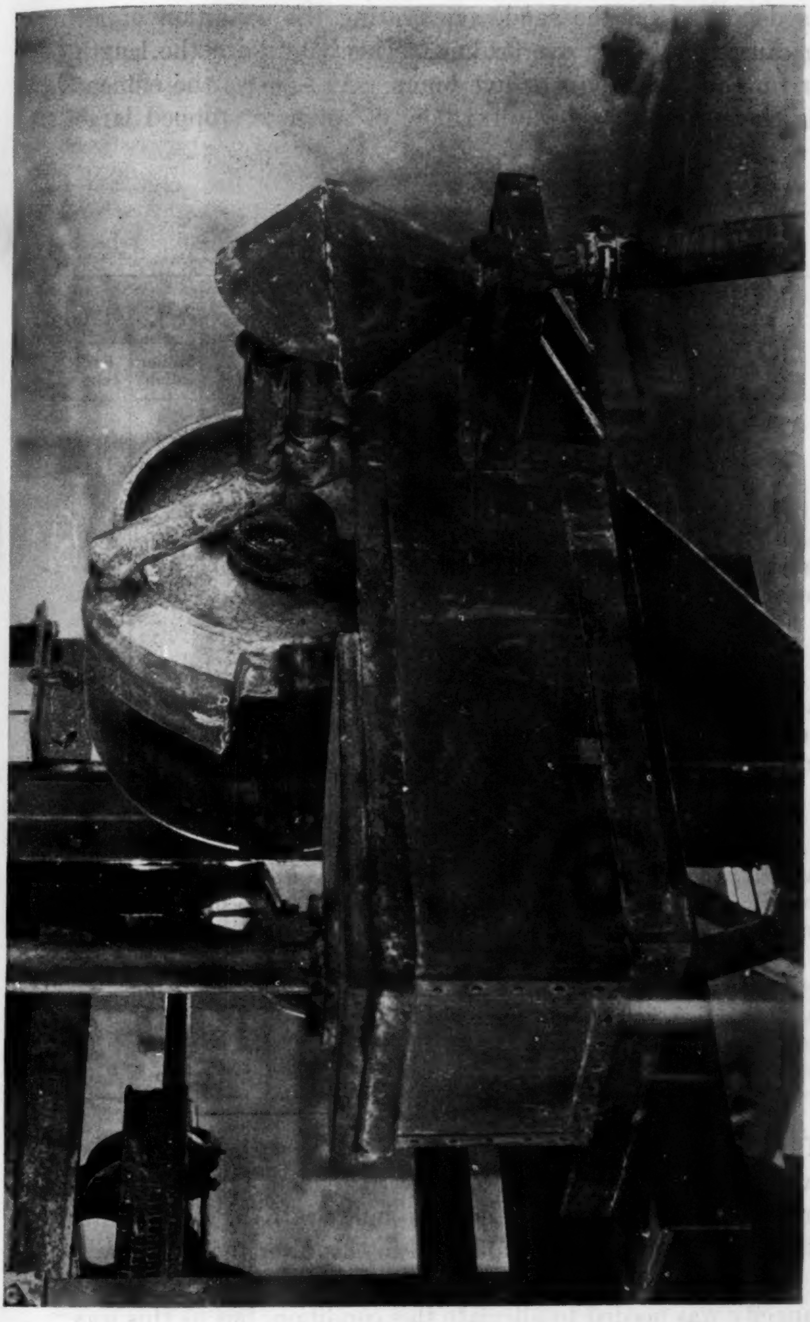


FIG. 4. ALUM DOSING WHEEL

Alum is lifted by air ejector to box, hard lead dippers on wheel dump into suction side of Schutte and Koerting water operated lead ejector which conveys alum to point of application. Speed is regulated by Reeves variable speed transmission.

algae is deposited on the sand, aggravating the condition of air binding caused by severe overloading. This cuts down the length of filter runs to about four or five hours. As soon as the effluent valve is closed and the downward flow of water is stopped large

TABLE 2

Filter operations

Monthly averages (1925–1926–1927)

	NUMBER FILTERS IN SERVICE	AVERAGE RATE FILTRATION	AVERAGE FILTER RUN	NUMBER FILTERS WASHED	NUMBER FILTERS JOLTED	AVERAGE RATE OF WASH	AVERAGE TIME OF WASH	WASH WATER	
								Million gallons	Per cent
		m.g.d.	*hours*			*inches per minute*	*minutes*		
1925:									
Total		8180.8						358.1	
Average	6.3	22.4	11:30	12.7	3.2	22.9	4.0	0.98	4.4
Maximum	8.0	34.7	25:00	25.0	31.0	24.0	7.0	1.9	9.6
Minimum	4.0	14.5	6.30	6.0	0	20.0	2.0	0.49	1.6
1926:									
Total		8877.0						307.5	
Average	7.4	24.4	34:36	11.2	1.6	24.0	4.3	0.87	3.6
Maximum	8.0	35.3	143:00	23.0	25.0	24.0	8.0	2.2	8.4
Minimum	7.0	13.8	7:30	0	0	24.0	0	0	0
1927:									
Total		8666.8						180.9	
Average	6.8	23.7	52:22	5.0	2.0	23.0	6.0	0.52	2.12
Maximum	8.0	33.8	132:00	14.0	22.0	29.0	15.0	1.19	4.6
Minimum	5.0	13.9	12:25	0	0	10.0	0	0.11	0
Average 3 years:									
Average	6.8	23.5	33:06	9.6	2.2	23.3	4.7	0.79	3.3
Maximum	8.0	35.3	143:00	25.0	31.0	29.0	15.0	2.2	9.6
Minimum	4.0	13.8	6:30	0	0	10.0	0	0	0

gobs of entrapped air break loose and rise to the surface displacing pea gravel and sand. In order to study the effect of air binding a standard scale experimental filter was built with a glass front. When the air conditions were duplicated in this filter the gravel was plainly seen to rise and allow the sand to drop down. Additional filter capacity was needed to alleviate this condition, but as this was

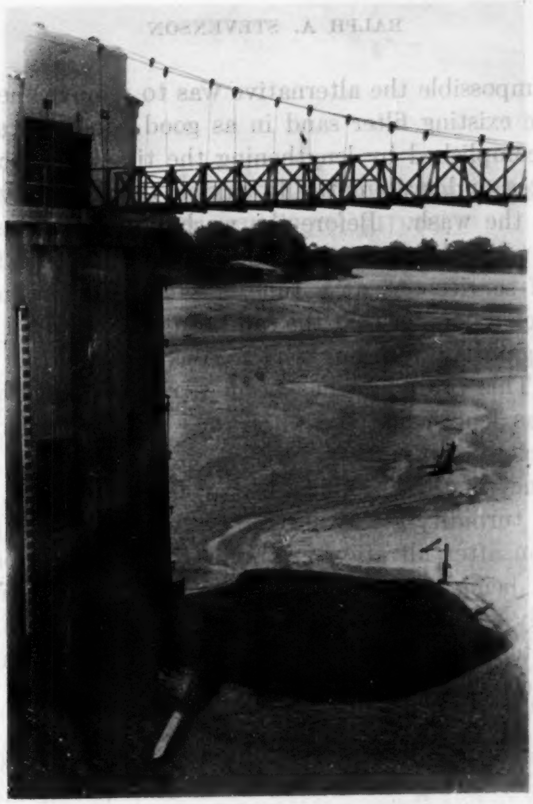


FIG. 5. INTAKE PIER SHOWING LOW AND HIGH WATER CONDITIONS

at the time impossible the alternative was to remove the mud balls and keep the existing filter sand in as good condition as possible. This was accomplished by lengthening the time of wash from four to six minutes and systematically raking the pea gravel back in place during the wash. Before the wash a hose was used on top of the sand which helped to break up mud balls; the filters were taken out of service one at a time, a sand ejector placed on the gutters in such a manner that the sand could be shoveled into it and discharged against a reclining screen thus cleaning the sand very effectively. This procedure has been carried out for the last two years and has lengthened the filter runs and prolonged the time when a complete overhaul was necessary.

The efficiency of the filters has been consistently high, and at no time has the turbidity of the filtered water exceeded 5 p.p.m.

Chlorination after filtration is practiced. A small dose, usually 0.15 to 0.20, being applied at the entrance to the storage basin. This is usually sufficient to give a residual test of 0.02 p.p.m. after four hours retention in the storage basin, although at times due to a high organic content a dose of 2 p.p.m. will leave but a faint trace of chlorine after four hours contact.

During the ensuing year we hope to run a plant scale experiment of heavily chlorinating the filtered water and de-chlorinating with sulphur dioxide.

Operation at the Sacramento Filtration Plant has been complicated by low water and high water with the attendant flashy conditions, but with the exception of a faint odor noticeable for about two months in the year, at no time has the water leaving the plant failed to meet, and in fact exceed, the standards of purity required.

CHEMICAL TREATMENT OF THE KANSAS CITY, MISSOURI, WATER SUPPLY[1]

By George F. Gilkison[2]

The Kansas City, Missouri, water department takes its supply from the Missouri River, a highly polluted surface supply, carrying a large amount of suspended matter. This suspended matter varies over a wide range, both in parts per million and in coefficient of fineness. The turbidity of this stream varies from 15 p.p.m. in the winter, when the river is frozen over, to 18,000 in the spring, when the ice breaks up. The coefficient of fineness of suspended matter varies from 0.8 to 1.5, based on the American Public Health Association standard.

The concentration of dissolved solids also fluctuates, varying from 300 in the spring when the river is high, due to snow water from the upper watershed and spring rains over the lower portion, to 540 p.p.m. when the river is at its lowest level in the winter. The Missouri River watershed covers approximately 430,360 square miles.

Very little dissolved coloring matter appears and at no time of the year is it a factor in computing the required chemical dose.

Ours is a comparably hard surface supply, the total hardness varying from 148 to 296 p.p.m., expressed in terms of calcium carbonate. The average total hardness for the past year was 219, of which 148 were due to calcium and magnesium bicarbonate, and 70 p.p.m. to calcium and magnesium sulphates.

The B. coli index of the Missouri River is very high at Kansas City, ranging from 50 to 100,000 per 100 cc. completely confirmed.

INDUSTRIAL WASTE

This division is indeed fortunate that, with all the industrial wastes discharged in the Missouri River, none of them, to date, has directly or indirectly interfered with our purification process. The characteristic odor and taste of chlorine which you have doubtless

[1] Presented before the Missouri Valley Section meeting, October 5, 1928.

[2] Chief Chemist, Water Department, Kansas City, Mo.

all noticed in our city water supply, cannot truthfully be charged to industrial waste, but is the direct result of trying to maintain open storage basins in our industrial districts.

The treatment at Kansas City consists of presedimentation, followed by coagulation with alum and lime, and disinfection with liquid chlorine.

PRESEDIMENTATION

With a raw supply carrying as high as 18,000 p.p.m. turbidity, it necessarily follows that presedimentation is an important step in purification here. At the Quindaro Purification Works the raw water is pumped into a 4,750,000 gallon basin, known as the horseshoe basin, or basin no. 1. It flows from this basin into basin no. 2 over a weir, in front of which is a stilling baffle. This basin has a capacity of 14,250,000 gallons, making a total of 19,000,000, which is four and one-half hours' retention, based on 100,000,000 gallon daily pumpage. These basins are flushed regularly, but nevertheless they soon fill up with sludge, necessitating their being cut out of service and washed. Three hundred and thirty thousand six hundred and sixty-five tons of suspended solids were removed from the water in the Quindaro basins last year, 298,241 tons of which were removed in the preliminary basins, that is, 90 per cent of the basin load was carried by the preliminary basins. At the North Kansas City Purification Works we have four 4,000,000 gallon circular preliminary basins equipped with Dorr mechanical clarifiers. This gives four hours retention, based on 100,000,000 gallon daily pumpage. This equipment provides for the continuous removal of the sludge, and eliminates the necessity of cutting units out of service for washing. The new North Kansas City Purification Works have been in service but a short time, and have been operating much below rated capacity. For this reason, no attempt will be made to quote efficiency at this plant.

APPLICATION OF CHEMICALS

At the Quindaro plant, the alum and lime are applied to the water through perforated lines as it passes over no. 2 weir, that is, as it passes out of the presedimentation basin. The water then enters a short mixing chamber, with a retention period not exceeding one minute. It then enters the coagulation basins, where it remains seventeen and three-tenths hours, based on 100,000,000 gallon daily

pumpage. It is chlorinated as it leaves these basins, and is pumped to the distribution station. No filters are provided at the Quindaro Purification Works. The average raw water turbidity for the past year was 3100 and the delivered water 10 p.p.m., or an average of 99.68 per cent removal by sedimentation and coagulation alone.

Approximately 5,000,000 pounds of alum were used by the Water Department last year, all of which was manufactured by the department. The total cost of manufacturing alum for the past year was $16.22 per ton, delivered to the storage bin. The dose of alum varied from 4.26 to 0.58 grains per gallon, the average over the year being 1.67. The lime used was lump quick lime, containing an average of 92 per cent water soluble calcium oxide. The dose used varied from 2.33 to 0.29 grains per gallon, the average for the year being 0.84.

The dose of lime is approximately one-half that of alum. This is the ratio we use most of the year. With an average hardness of 219 p.p.m., it seems the addition of lime, purely for aiding coagulation, would be unnecessary, but experience has shown us that a much smaller dose of alum may be used, if approximately one part of lime is used to two parts of alum.

CHEMICAL COST

The chemical cost of purification for the past year was $2.94 per million gallons, delivered to the city. This is the cost of chemicals alone, and does not include handling cost.

In view of the marked contrast in methods and equipment at the Quindaro and North Kansas City Purification Works, it is indeed unfortunate that enough operating data on the new plant are not at hand for comparison of efficiency and cost, but it is impossible to compute the efficiency of a hundred million gallon daily plant from the results obtained by operating it at one fourth rated capacity.

COLON BACILLI IN PRESSURE TANK WATER SYSTEMS

By W. L. Mallmann[1]

The presence of the colon bacillus in a water supply, as demonstrated by the American Public Health Association's Standard Methods of Water Analysis, is quite generally accepted as an indication that the water is unsafe for drinking purposes. Many laboratories, however, recognize the fact that the above mentioned methods do not exclude Aerobacter aerogenes which is generally considered to be a non-fecal strain that has no sanitary significance. Such laboratories condemn only water supplies containing the so-called fecal strains of the colon bacillus. This latter procedure is certainly the better where the flora of the water are known and frequent examinations are made.

For a number of years, the writer has been dealing with rural water supplies where one or rarely two or more samples are obtained from each supply. Complete histories accompany these samples in all instances, but the writer has often questioned his right to condemn a water supply by the presence of even the so-called fecal colon bacilli. In many instances, Escherichia coli has been isolated from wells the history of which would make the presence of fecal contamination impossible. The writer wishes to report in this paper one condition frequently encountered in which the presence of the colon bacillus has no sanitary significance.

The installation of pressure water systems on the farms has increased very rapidly during the past ten years. These systems placed in the basement of the house consist of a large pressure tank into which air is pumped to force water to the various parts of the house and other buildings. The installations of such systems in rural homes, if they have any effect, should improve the quality of the water as improved wells are generally installed along with the pressure system. All of the wells studied by the writer were either deep

[1] Department of Bacteriology, Michigan State College, East Lansing, Mich.

driven wells or drilled wells. Such wells should be free from any of the colon bacilli, either Escherichia coli or Aerobacter aerogenes, but these bacteria have been found in pressure tanks.

One instance might be presented in explanation of such cases. A sanitary analysis of a sample of water was made from a consolidated school well. The results of the tests were as follows: Strong gas production in all five 10 cc. portions of water examined in twenty-four-hour incubation at 37°C. The total bacterial count at 37°C. for twenty-four hours was 300 colonies per cubic centimeter of water. Eosin-methylene blue medium was positive for Escherichia coli. The isolated colonies fished gave strong gas production on lactose broth, positive methyl red test and negative Voges-Proskauer test. Since the organism present was undoubtedly Escherichia coli, the school was notified not to use the water and a second sample was requested along with a complete description of the water supply. The results on the second sample were approximately the same as on the first, thus confirming the presence of Escherichia coli. The description of the supply revealed a deep drilled well of proper construction and in excellent repair with no evident source of pollution. The water was piped directly into the school house where it entered a pressure storage tank.

On the strength of this information, two samples were requested, one from a tap in the building and the other directly from the well. The results obtained from the sample taken from the tap were similar to the two previous samples, while the sample from the well was free from colon bacillus in the five 10 cc. portions tested. The total count was only 9 colonies per cubic centimeter of water.

The data indicated that the well water was free from the colon bacillus, thus demonstrating that the contamination was in the pressure tank.

Similar cases have been found in similar pressure systems in rural homes. In most cases, however, the organism isolated has been Aerobacter aerogenes.

The treatment recommended for disinfecting the pressure tank and piping has been to empty the tank, add a strong solution of hypochlorite of lime (5 p.p.m. free chlorine), refill the tank and allow to stand for several hours. All pipes are flushed with chlorinated water and the tank is again emptied. This treatment has in all instances removed the colon bacillus and reduced the total count to an unsig-

nificant number. None of the systems treated have ever become recontaminated.

No explanation for the entrance of the colon bacillus is offered. Only two means of pollution seem possible: (1) contaminated tank and piping and (2) entrance of the organism through the suction line on the air compressor.

These data are presented merely to call to the attention of the water bacteriologist, who is testing rural water supplies, that too much stress should not be placed upon the laboratory test, as cases arise where the presence of Escherichia coli does not indicate sewage pollution and does not indicate a dangerous source of water. Rural water supplies should never be condemned on the results of one test unless the description of the well clearly indicates an improperly constructed well.

OPEN AND COVERED RESERVOIRS AT WASHINGTON, D. C.[1]

By Carl J. Lauter[2]

The city of Washington, D. C., which uses the Potomac River as its water supply, was up to 1928 divided into four distinct areas of distribution. The gravity supply is in that district lying below the level of the filtered water reservoir at the McMillan slow sand plant and is principally the business district. Direct pumpage reaches Capitol Hill and the high area to the north of gravity up to 140 feet elevation. The area between 140 and 210 feet is served by the two Brightwood reservoirs, two open basins of approximately eight million gallons capacity each. The highest ridge and outlying districts are served by the covered Reno reservoir at 425 feet elevation and by a standpipe a little above this. It is upon the last two mentioned reservoirs that the comparisons are to be made in the following discussions.

It is the purpose of this paper to give some comparative figures on the quality of water as found in the reservoirs and to draw conclusions from these data in regard to open and covered reservoirs as operated under similar conditions.

All water passes through the same filtration plant where it is chlorinated before reaching any distribution system or mains. It also all passes through the gravity area covered reservoir at the McMillan plant on its course to the District of Columbia pumping station, a few hundred yards below the former basin and plant from where it is pumped to the higher services. In the figures all data for filtered water will therefore be the same for the gravity and for direct pumpage areas and at times for other areas when consumption is less than pumpage.

[1] Presented before the Water Purification Division, the San Francisco Convention, June 14, 1928.

[2] Chief Chemist, Washington Filtration Plants, Washington, D. C.

These figures cover a period of four years from 1923 to 1927, being monthly averages of weekly tests on the two reservoirs and monthly averages of daily tests on the filtered water for bacteria. The microscopic and chemical tests are on weekly samples.

Both of the reservoirs in question are of concrete construction, the Brightwood or open one being located in the heart of the best residential section about 3 miles from the filtration plant and between a main avenue and directly adjacent to the densely wooded Rock Creek Park. The average daily service from here is about fifteen million gallons. The basins are arranged with flumes extending to diagonal corners from influent end with double check valves at influent point to facilitate circulation, and so arranged to take water at two elevations if desired.

The covered five million gallon Reno reservoir is a smaller one having a flat or slab roof, upon which there are public tennis courts. This basin is about 5 miles from the pumping station and serves about ten million gallons daily.

BACTERIOLOGICAL EXAMINATION

From figure 1 it will be noted that the curves for the filtered water, which was chlorinated to the extent of 0.15 p.p.m. and at intermittent periods, is very close to that for the covered reservoir. The exceptionally high peak on the covered reservoir in the summer of 1925 was due to the tearing up of and installation of new mains on this service. The decided drop in July of the same year came as a result of chlorination of the reservoir which practice was continued at both reservoirs after this date whenever the bacteria began to increase.

Up to September, 1924, both open basins were continuously in service except for purposes of cleaning which was done two or three times a year, the sides being scrubbed down with hose and sprayed with copper sulphate solution. After this date, however, only one basin was in continuous service, alternating with each other when cleaning was necessary. Thus the circulation was virtually doubled and the results were highly beneficial, as will be noted on figure 1.

The greater distance of the covered reservoir from the pumping station would be a factor reacting unfavorably against it, other things being equal, but its smaller capacity of five million gallons allowed better circulation. This fact seems to have been demonstrated by the reactions of the water in the basins. The pH, alkalinity, etc. lagged in the case of the covered reservoir by two or three days,

while in the open ones it was nearer two weeks over these reactions in the filtered water at the McMillan plant.

The net result of the bacterial figures is in favor of the covered reservoir, the averages for the four years being a count of four on the filtered water, with seven plus for the covered basin, an increase of only 90 per cent, while in the open the average count was forty-four, an increase of 1000 per cent. The open basins also showed positive B. coli tests in a greater percentage of tubes and in smaller dilutions. In all cases five 10 and five 1 ml. dilutions were made on each of weekly and on daily filtered water.

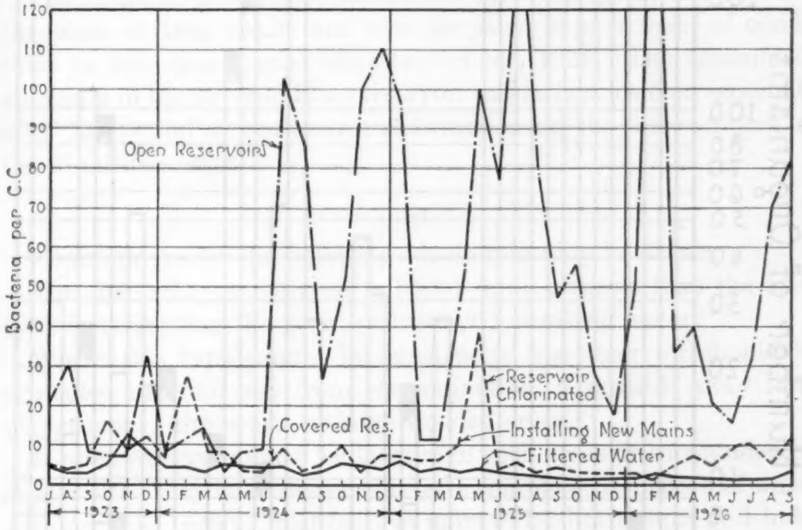


Fig. 1. Changes in Bacterial Counts

MICROSCOPIC EXAMINATION

In the weekly microscopical examination, even more conclusive evidence in favor of the covered basins is at hand. The organisms found were grouped in two classes; Synedra and all other organisms. As was to be expected, the effluent from the slow sand filters was entirely free from either group for the period, while in the covered reservoir there were only three instances when organisms were noted. All months of the year showed the presence of organisms in the open basins with large increases in the summer months.

After August, 1924, there is a noticeable falling off of number of

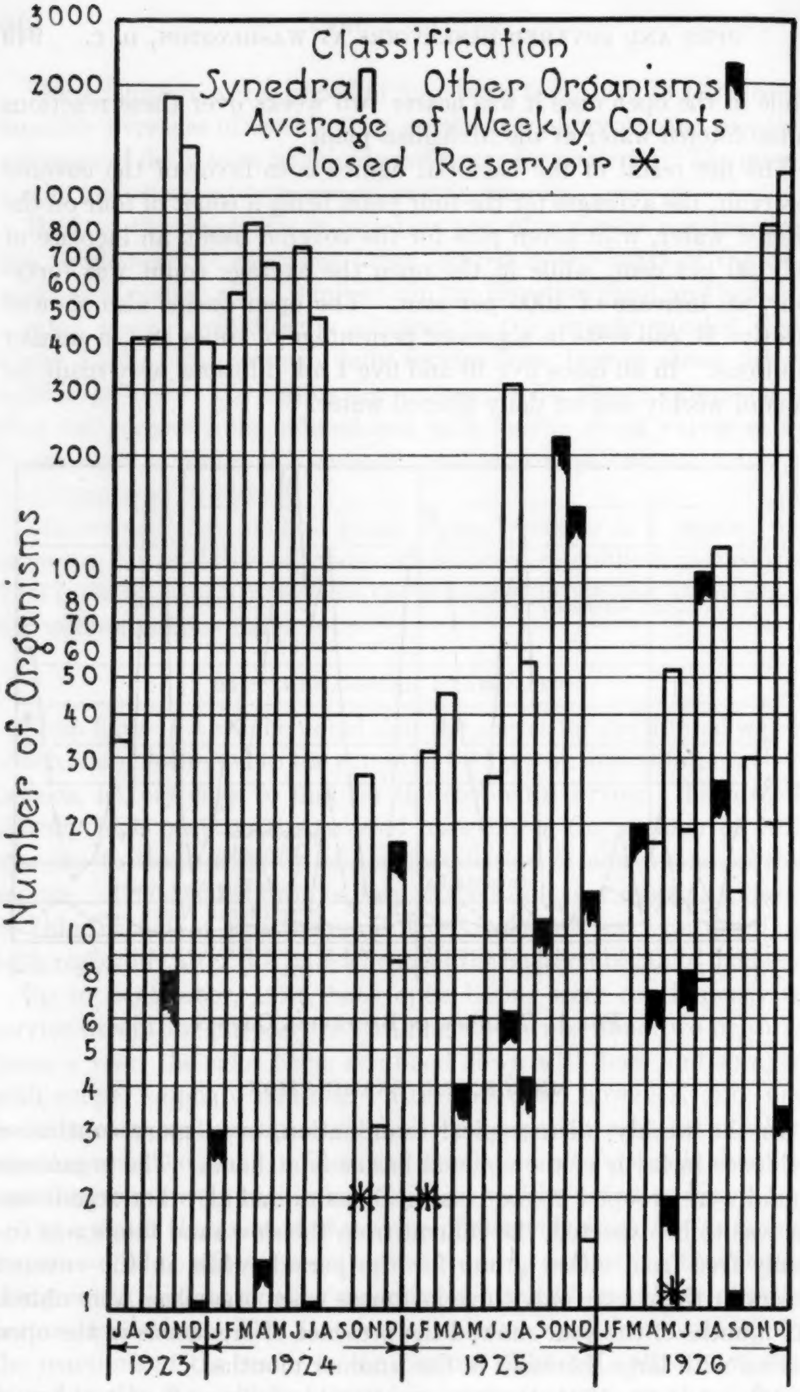


Fig. 2. Open Reservoir Microorganisms

organisms in the open basins caused by the aforementioned removal from service of one of the two twin open reservoirs and putting them into intermittent service.

During all of this period of study it may be interesting to note that no algae or microscopic organisms manifested themselves to such a degree in the three large preliminary sedimentation reservoirs of the system prior to filtration to require treatment of these reservoirs. All of the offending organisms were therefore blown into the open basins from the adjacent tennis courts and from falling leaves from the park just beyond the courts. These findings are shown in figure 2.

The albuminoid ammonia and nitrate curves in figures 3 and 4 show the same striking results and also the same improvement of conditions in the open basins after September, 1924. The albuminoid ammonia in the covered Reno reservoir was in fact lowered especially after the period of continuous chlorination on the filter effluent at the McMillan plant.

CONCLUSIONS

From our results the following conclusions may be drawn.

The covered type reservoir is better in all respects than the open type in preserving the good qualities of a drinking water.

Where this type cannot be constructed the reservoir should be protected in some way from contamination, if possible away from playgrounds, dirt courts or large wooded areas.

If open reservoirs must be located in such areas or surrounded by streets, etc., they be so designed to give complete circulation of water with no dead corners, and built in pairs to facilitate rapid and independent cleaning on short notice.

Chlorination must at times be resorted to at those reservoirs, especially if large new services are installed to said basins.

Sampling and testing of reservoirs is as important if not more so than daily routine on filter effluent only.

SUPPLEMENTARY DATA

Since these data were completed and the results were tabulated we had occasion to make another series of studies upon a large sedimentation reservoir over a short period of time.

During March and April, 1929, the McMillan slow sand plant was out of service, all of the water being filtered and distributed from

the Dalecarlia rapid sand plant, situated just at the effluent end of the first large 300 million gallon sedimentation reservoir, 9 miles from the

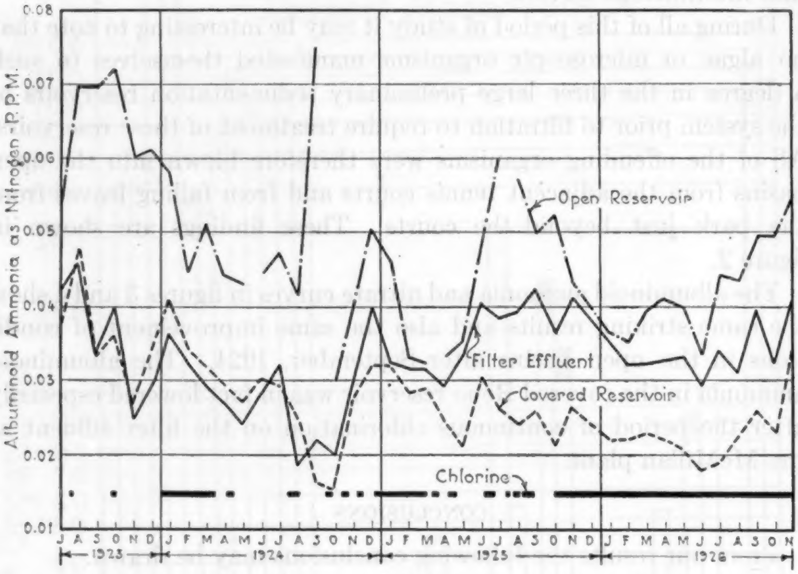


FIG. 3. CHANGES IN ALBUMINOID AMMONIA

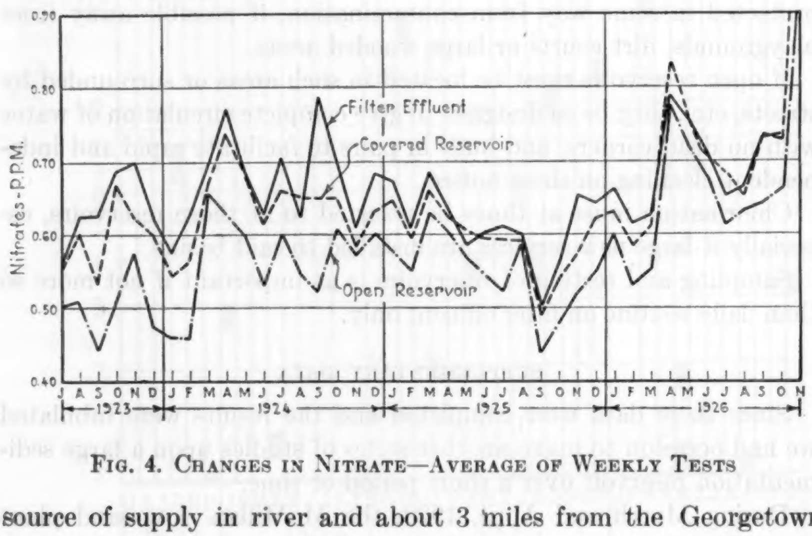


FIG. 4. CHANGES IN NITRATE—AVERAGE OF WEEKLY TESTS

source of supply in river and about 3 miles from the Georgetown Reservoir, which is about 36 acres in area and of about 175 m.g.

capacity. This is normally the second sedimentation reservoir for the McMillan Plant and is so constructed and divided by dams to include a sedimentation chamber of about one-eighth the area. This is used as a coagulating or settling basin when turbidities are 70 to 80 and alum is added to assist in the chlorination of water.

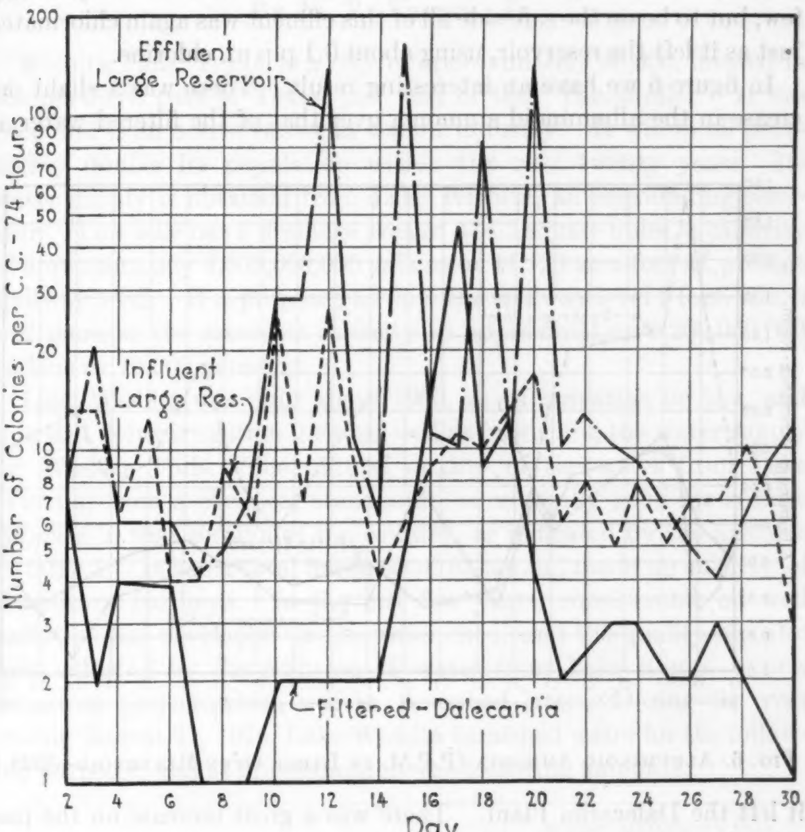


FIG. 5. GEORGETOWN RESERVOIR—BACTERIAL COMPARISON—APRIL, 1928

During March this reservoir was supplied with filtered and chlorinated water to displace the raw water and in about three weeks following the first introduction of such water it was pumped from the effluent end directly to the city.

It was upon this large reservoir with a continuous circulation of 40 m.g. daily that the data in figures 5 and 6 were obtained.

There is a great increase in bacterial count at the influent point, due to rapid flow through the 3-mile 8-foot circular conduit, which for past years delivered only raw water, but there is again a decided increase in count over this figure at the outlet point, or an average of 250 per cent in the course of travel through the reservoir in four to five days' time. The number of positive colon tests was surprisingly few, but to be on the safe side all of this effluent was again chlorinated just as it left the reservoir, using about 0.1 p.p.m. chlorine.

In figure 6 we have an interesting result. There was a slight decrease in the albuminoid ammonia over that of the filtered water as

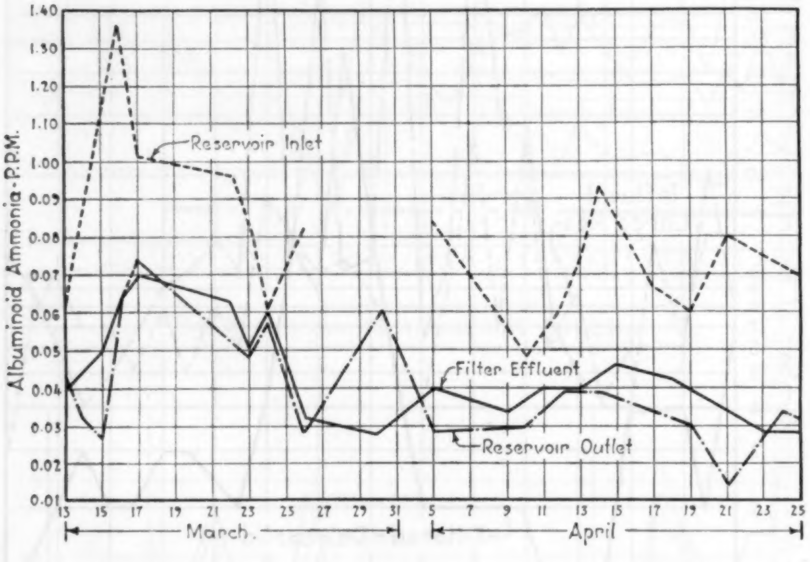


FIG. 6. ALBUMINOID AMMONIA (P.P.M. IN LARGE OPEN RESERVOIR—1928.)

it left the Dalecarlia Plant. There was a great increase on the passage through the conduit, but this was entirely oxidized on its course through the basin. This is doubtless due to the good circulation and the fact that ratio of surface area to volume is greater in this reservoir than in the smaller distributing open basin of the first series of results.

There was no free chlorine in the filtered water leaving Dalecarlia, all of oxidation taking place in the open basin.

The two open Brightwood Basins are now dry and out of service permanently, being supplanted by two new concrete reservoirs served by the Dalecarlia Plant. The McMillan slow sand plant serves only the old gravity area, about 40 per cent of the entire supply of Washington.

PROTECTION OF AN IMPOUNDED WATER SUPPLY FROM OIL FIELD DRAINAGE AND IRRIGATION WATER[1]

By N. T. Veatch, Jr.[2]

Wichita Falls, Texas, is a city of approximately 60,000, located in the northern part of Texas, just east of the "Panhandle." It has been and is an active center in the oil industry. It will in all probability double its population within the next twenty years. Its water supply is obtained from Lake Wichita, an impounding reservoir. This lake has a drainage area of 134.3 square miles, a capacity of approximately 4,600,000,000 gallons or 14,120 acre-feet at present spillway level. It is proposed to raise the spillway level 2 feet, which will increase the reservoir capacity to approximately 6,200,000,000 gallons or 19,000 acre-feet.

Lake Wichita was built about 1900, as an irrigation project, and it is still doing irrigation duty, as well as furnishing the water supply for Wichita Falls. The natural quality of the water from Lake Wichita, from a chemical standpoint, is unusually good for surface supplies in that section of the country, as indicated by the analysis in table 1. It has a total hardness of 65 p.p.m., made up entirely of temporary hardness. In the last few years, considerable oil well pollution has developed on the water shed, and the quality has also been affected by the addition of water from Lake Kemp, a new irrigation project which will be described later. During the year ending August 31, 1926, Lake Wichita furnished water for the following purposes, and at approximately the following amounts:

	BILLION GALLONS PER YEAR
Irrigation	1.96
Water supply	1.15
Waste in ditches and in irrigation	1.50
Total	4.61 or 12.7 million gallons per day

[1] Presented before the Water Purification Division, the San Francisco Convention, June 14, 1928.

[2] Of Black and Veatch, Consulting Engineers, Kansas City, Miss.

This amount represents the amount used for several preceding years. The average per capita rate of water used in the city is 66.3 gallons, based on an average population of 50,000 during the period analyzed, with a maximum of 117 gallons per day. The maximum hourly rate was 11 million gallons per day, or 332 per cent of the yearly and 248 per cent of the monthly averages.

About five years ago, Lake Kemp was built as a source of supply for an irrigation project. It was considered also as an additional water supply for the city. Lake Kemp has a drainage area of 2650 square miles, a water surface of 31 square miles, or 20,000 acres, and a capacity of 500,000 acre-feet, or 163 billion gallons. A large part of the drainage area is covered with "gyp" deposits which are reflected in the quality of the water, detailed analysis of which is given later. As a comparison with the Lake Wichita

TABLE 1
Chemical analysis of Lake Wichita water
Results in parts per million

DATE OF SAMPLING	TOTAL SOLIDS	TEMPORARY HARDNESS	PERMANENT HARDNESS	TOTAL HARDNESS	SODIUM	BICARBONATES	SULPHATES	CHLORIDES
March, 1922	480	65	0	65	26.2	78	19	16.5
January, 1926	1,136	81	437	518	227	98.6	432	410.0
September, 1926	878	82	252	334	135	100.0	169.0	262.0

water under natural conditions, having a total hardness of 65 p.p.m. and made up entirely of temporary hardness, the Lake Kemp water, as represented by the ditch water below the diversion dam, had in April, 1924, a temporary hardness of 71 p.p.m. and a permanent hardness of 578 p.p.m. This water, as will be shown later, is excellent for irrigation purposes, but unfit for domestic use, even if softened.

The City of Wichita Falls, therefore, has two sources of water supply, one, Lake Wichita, of excellent quality if protected from outside contamination, and which, if conserved, will serve the city probably fifteen years at least, and the other, Lake Kemp, of unlimited quantity but of a chemical quality unfit for city use. Therefore, its water supply problem hinges on the possibility of protecting the soft supply from Lake Wichita from pollution by the oil well development, and from any ill effects from the irrigation operations on its water shed.

The possibility of constructing a dual distribution system was studied carefully, as it was thought that it might be feasible to build a low pressure system that would supply the city with lawn irrigation and street flushing service, using Lake Kemp water. A careful analysis of this plan showed quite clearly that it was economically impractical, at least until the soft water supply from Lake Wichita has reached the limit of its capacity. When this time comes, the city can either construct a dual system or go after more soft water. As the latter plan must be followed eventually, if the city continues to grow, it will probably be found advisable to get more soft water, as this can be done on an adjoining water shed, at a reasonable cost. The dual system would probably extend the useful life of Lake Wichita only about ten or fifteen years. As a high pressure system will always be required for fire service, the most logical plan to follow, in case a dual system is attempted, would probably be to turn the present system over to use for fire, lawn irrigation, street flushing service, etc., and build a new system for drinking, household and industrial uses. This would require two high pressure systems, but the large sizes required for irrigation and fire would be combined in one. At any rate, it was unnecessary to consider the question at this time. When the question of additional soft water supply must be decided, it will be almost entirely a question of economic balance between the cost of a dual system and the cost of developing additional supply. There are, in addition to the cost, a number of things which make a dual system questionable. Operating costs, maintenance, etc. will be much greater, and the house owners will be forced to additional plumbing expense. The safety from a health standpoint is also certainly questionable. Danger of cross connections, children drinking from the sprinkling system, etc., would make the dual plan questionable, even if costs should balance.

As has been pointed out, Lake Wichita at present is supplying, approximately, three times the amount of water used by the city. The city supply is taken from a main irrigation ditch about 7 miles below the Lake. It is planned to build a 42-inch flow line from the lake to the city plant, and to supplement the irrigation duty, now supplied by Lake Wichita, with water from Lake Kemp, thus conserving all of the former supply for city use.

POLLUTION OF LAKE WICHITA

In addition to conserving the supply, it is also necessary to protect its quality. The protection of Lake Wichita from pollution by salt water from oil development, and from hard water from irrigation operations, offers a rather unusual problem in water-shed protection.

The pollution in this instance is of inorganic nature and the object of the protective measures is the conservation of the natural softness and sweetness of the water, rather than of its sanitary quality.

Approximately one-third of the drainage area of Lake Wichita will eventually be irrigated with water which is saline and hard. The waste water and drainage from this irrigation project will be discharged into the lake. Furthermore, there are in the neighborhood of 500 producing oil wells on this drainage area, from which, approximately, 1200 barrels or 50,400 gallons of salt water are pumped daily. This salt water is at present either stored in salt water ponds for evaporation, or discharged into the country drainage, which means that it eventually reaches Lake Wichita.

It is proposed to protect the lake from pollution from these sources by the construction of a girdling ditch of sufficient capacity to carry flow, including waste water from the irrigated land, to the stream at a point below the dam which forms Lake Wichita. This procedure seems a rather obvious expedient, but some of the data secured in the study which led up to its adoption are interesting and will be discussed below.

Lake Wichita impounds water from a drainage area of rather flat country, which, with the exception of the irrigated portion, is uncultivated, and covered by a rather sparse growth of mesquite, chaparral and prickly pear.

Previous to the drilling of oil wells on this area and to the use of Lake Kemp water, the water in Lake Wichita was unusually soft for impounded water in this locality as shown in table 1.

The increase in hardness and salinity is due in part to the oil well drainage, in part to the fact that, during 1925, the lake was partially filled from Lake Kemp to make up for water used for irrigation below Lake Wichita, and, in part from waste irrigation water on the Lake Wichita drainage area.

The fact that the added solids are in the nature of permanent hardness and salinity, naturally directed the protective measures toward the source of the pollution, rather than toward treatment of the

water, since the softening of this water would be expensive and unsatisfactory. The problem, therefore, resolved itself into a study of the salt water and irrigation situation.

The duty of irrigation water in this section has not been definitely determined, but indications are that it will be in the neighborhood of

TABLE 2

Chemical analysis of irrigation water from Lake Kemp

Results in parts per million

Total solids	1,994
Bicarbonates	86.6
Chlorides	523.0
Sulphates	481.4
Calcium	208.0
Magnesium	32.0
Sodium	304.0
Temporary hardness	71.0
Permanent hardness	578.0
Total hardness	649.0

TABLE 3

Effect on runoff, with all hardness and salinity reaching Lake

Results in parts per million

Salinity expressed as sodium chloride	
Year of maximum rainfall	465
Year of average rainfall	1,027
Year of minimum rainfall	5,714
Hardness expressed as calcium carbonate	
Year of maximum rainfall	395
Year of average rainfall	865
Year of minimum rainfall	4,396

24 inches per year, or 66,000 acre-feet for the portion of the drainage area to be irrigated.

The character of the irrigating water is shown in table 2.

Based on the above analysis and on the yearly use of 66,000 acre-feet of this water for irrigation, approximately 69,200 tons of sodium chloride and 58,200 tons of total hardness, expressed as calcium carbonate, will be brought into the drainage area each year.

The effect on the run-off, provided all the hardness and salinity of

the irrigation water reaches the lake either by drainage or by seepage, is shown in table 3.

Such a condition would represent a maximum in each case to be reached only in case all of the chemicals were evaporated out of the irrigating water and washed off by the rains, but the above figures are given as a yard stick to indicate the possible effect of this amount of salinity and hardness.

Similarly, the character of the salt water produced from the wells is shown in table 4.

TABLE 4

Chemical analysis of salt water from oil wells

Results in parts per million

Total solids	167,330
Calcium	9,550
Magnesium	1,770
Sodium	46,553
Bicarbonates	11
Chlorides	93,600
Sulphates	0
Temporary hardness	9
Permanent hardness	30,871
Total hardness	30,880

Based on a daily production of 50,400 gallons, the following amounts of chemicals will be introduced into the drainage area each year:

	tons
Sodium chloride	9,057
Calcium chloride	2,001
Magnesium chloride	512

The salinity and hardness of the runoff will be affected as shown in table 5.

Summarizing the above, it is seen that a total of, approximately, 78,300 tons of sodium chloride and 60,500 tons of hardness, expressed as calcium carbonate, will be introduced into the drainage area each year, and must be absorbed into the soil or diverted around the lake, if the water is to be kept in a usable condition.

It has been amply demonstrated in a number of irrigation districts that little or none of the salts of the irrigating water will be

washed off by rains into the country drainage, provided the ground-water level is far enough below the surface of the soil to prevent the highly-concentrated soil-solution from being brought to the surface by capillary action and there evaporated.

TABLE 5

Effect of oil well waste if allowed to reach Lake

Results in parts per million

Salinity expressed as sodium chloride	
Year of maximum rainfall	61.0
Year of average rainfall	134.0
Year of minimum rainfall	748.0
Hardness expressed as calcium carbonate	
Year of maximum rainfall	16.0
Year of average rainfall	35.0
Year of minimum rainfall	93.5

TABLE 6

Results of soil analyses of irrigated and unirrigated land on the Bridwell Farm

DEPTH	WATER SOLUBLE MATERIAL		EFFECT OF IRRIGATION IN PER CENT OF SOLUBLE SALTS	
	Unirrigated	Irrigated		
feet	*per cent*	*per cent*		
1	0.070	0.080	0.01	Increase
2	0.230	0.085	0.145	Decrease
3	0.490	0.200	0.290	Decrease
4	0.290	0.190	0.100	Decrease
5	0.210	0.240	0.030	Increase
6	0.210	0.170	0.040	Decrease
7	0.180	0.210	0.03	Decrease
8	0.200	0.210	0.01	Decrease
9	0.210	0.170	0.04	Decrease
10	0.150	0.180	0.03	Increase

The soil in this district is a clay loam and sufficiently permeable to absorb the water, and the ground-water level is, at present, far enough below the surface to prevent capillary connection. The question, therefore, is, will it remain so after extended use of hard and saline irrigating water?

It is well known that all soils, particularly clay soils, have base exchange properties similar to the familiar Zeolite materials, and it

has been demonstrated by Schofield[3] and others that, when this base exchange results in adding sodium salts to the soil with the release of calcium salts, the permeability of the soil is reduced, and, conversely, when calcium salts are added to the soil, the permeability increases. This indicates that water in which the calcium salts exceed the sodium salts will tend to produce permeable conditions.

The ratio of sodium and potassium salts to calcium and magnesium in the Lake Kemp water is practically unity and, therefore, little, if any, base-exchange activity can be expected. This is borne out by tests made on samples from different depths taken from unirrigated land and from land irrigated from Lake Kemp. Both borings were made on the same farm in similar soils. Results of tests are shown in table 6.

These results indicate that there is little difference in the water-soluble material in the unirrigated and the irrigated soil, and that water-soluble salts have been leached out rather than increased by the Lake Kemp water.

The effect of irrigation, therefore, can be limited to the water which is lost by overflow of ditches or wasted by over-irrigation, which amount should, with careful conservation of water, vary from 10 to 20 per cent of the total used, or from 6600 to 13,200 acre-feet per year.

Since this amount of waste may be easily handled by a small ditch around the lake, this method was selected as being the logical solution of the problem and a ditch has been designed.

The salt-water problem in this field as elsewhere, seems to be far from solution. It is thought here that the most feasible scheme for disposing of the salt water is to allow the storage ponds to be emptied during dry weather, when the flow can be intercepted by the girdling ditch.

[3] Journal of Agricultural Research, xxi, pages 265, 271, 1921; xxvii, no. 9, March, 1924.

TYPHOID FEVER IN THE LARGE CITIES OF THE UNITED STATES IN 1928[1,2]

This report concerns the same eighty-one cities of more than a hundred thousand population that were discussed in the report for 1927.[3]

Four of the twelve New England cities (table 1) had no typhoid deaths during 1928; this is the largest number of cities in a single group ever to make such a record. For New Haven and Springfield it is the second successive year; Lowell also had a clear record once before (in 1925). Eight of the cities have 1928 rates below 1.0 (as against six such cities in 1927). Four of the cities with 1928 rates below 1.0 had similar rates in 1927; Springfield and Bridgeport have had rates below 1.0 for the last three years. Boston's rate (0.6) is the lowest ever recorded for that city (1.0 in 1923 was its previous low point). The three cities which had the highest 1927 rates in this group (Lowell, Worcester and New Bedford) have conspicuously lower rates in 1928, all below 1.0. Though Fall River's 1928 rate (4.5) is higher than any rate in the New England group in 1926 and higher than the 1921–1925 average of any New England city, it is the only city in the group with a rate of over 2.0, whereas in recent years there have always been three or four cities with rates over 2.0. The group average (0.90) for the New England cities (table 12) for 1928 is

[1] Reprinted from the Journal of the American Medical Association, 92: 20, May 18, 1929, p. 1674.

[2] The preceding articles were published, Jour. Amer. Med. Assoc., May 31, 1913, p. 1702; May 9, 1914, p. 1473; April 17, 1915, p. 1322; April 22, 1916, p. 1305; March 17, 1917, p. 845; March 16, 1918, p. 777; April 5, 1919, p. 997; March 6, 1920, p. 672; March 26, 1921, p. 860; March 25, 1922, p. 890; March 10, 1923, p. 691; February 2, 1924, p. 389; March 14, 1925, p. 813; March 27, 1926, p. 948; April 9, 1927, p. 1148; May 19, 1928, p. 1624.

[3] The deaths from typhoid in each city are those reported to us by the respective health departments. The rates have been calculated on the basis of the midyear 1928 population as estimated by the United States Bureau of the Census. In ten instances in which such estimates were not available, the midyear population figures were furnished by the health departments of the respective cities. In one instance, when the health department was not heard from, the rate was figured on the basis of the 1927 midyear population.

the lowest ever recorded for New England and also the lowest in the country. The New England rate was lowest also in 1926 and 1927.

Two cities of the Middle Atlantic group (table 2) had no typhoid deaths in 1928—the same number as in 1927, one less than in 1926. It is the third consecutive year without a typhoid death for Yonkers and the fifth such year in its history (cf. 1920, 1922).

Of the eighteen cities in this group, nine have lower rates in 1928 than in 1927, and six of these nine have had a continuous decline beginning with 1926. Among the latter is Philadelphia, whose 1928 rate of 0.8, the lowest in its history and less than half of its 1926 rate,

TABLE 1

Deaths in cities in New England States from typhoid per hundred thousand population

	1928	1927	1926	1921–1925	1916–1920	1911–1915	1906–1910
Lowell	0.0	2.6	0.9	2.4	5.2	10.2	13.9
Lynn	0.0	0.9	3.8	1.6	3.9	7.2	14.1
New Haven	0.0	0.0	2.2	4.4	6.8	18.2	30.8
Springfield	0.0	0.0	0.7	2.0	4.4	17.6	19.9
Worcester	0.5	3.1	0.5	2.3	3.5	5.0	11.8
Boston	0.6	1.1	1.8	2.2	2.5	9.0	16.0
Bridgeport	0.6*	0.6	0.6	2.2	4.8	5.0	10.3
New Bedford	0.8†	3.3	0.7	1.7	6.0	15.0	16.1
Hartford	1.2	1.8	1.2	2.5	6.0	15.0	19.0
Cambridge	1.6	0.8	4.9	4.3	2.5	4.0	9.8
Providence	1.7	0.4	0.7	1.8	3.8	8.7	21.5
Fall River	4.5	2.3	0.8	2.3	8.5	13.4	13.5

* Population figures furnished by the health department: 155,000.
† Population figures furnished by the health department: 121,629.

bears comparison with the rates of Cleveland (0.6) and Chicago (0.5). Of the nine cities constituting the lower half of the table in 1928, six were also in the lower half in 1927. The 1928 rates of Pittsburgh, Camden and Erie are higher than any of the 1927 rates in this group. Camden, after a 1927 rate (0.7) which was far the lowest in its history (the next to the lowest being the 1919 rate of 2.7), reverted in 1928 to its old level with a rate of 4.4. Erie, after low rates in 1926 and 1927 (0.8 and 1.5), has a 1928 rate of 5.3, double its 1921–1925 average (2.3) and almost as high as its 1916–1920 average (6.9). It is the highest rate that has occurred in the Middle Atlantic group since

TABLE 2

Deaths in cities in Middle Atlantic States from typhoid per hundred thousand population

	1928	1927	1926	1921–1925	1916–1920	1911–1915	1906–1910
Elizabeth	0.0*	0.9	1.0	2.4	3.3	8.0	16.6
Yonkers	0.0	0.0	0.0	1.7	4.8	5.0	10.3
Syracuse	0.5	1.5	1.1	2.3	7.7	12.3	15.6
Jersey City	0.6	1.2	1.9	2.7	4.5	7.2	12.6
Albany	0.8	3.3	0.0	5.6	8.0	18.6	17.4
Philadelphia	0.8	1.4	1.9	2.2	4.9	11.2	41.7
Rochester	0.9	1.2	4.0	2.1	2.9	9.6	12.8
Newark	1.0	1.3	1.5	2.3	3.3	6.8	14.6
New York	1.4	1.3	1.9	2.6	3.2	8.0	13.5
Paterson	1.4	0.0	1.4	3.3	4.1	9.1	19.3
Scranton	1.4	0.7	4.2	2.4	3.8	9.3	31.5
Utica	1.9	2.9	0.0	3.9†			
Trenton	2.1	1.5	2.2	8.2	8.6	22.3	28.1
Buffalo	2.3	2.4	5.0	3.9	8.1	15.4	22.8
Reading	2.6	1.7	2.6	6.0	10.0	31.9	42.0
Pittsburgh	3.4	1.9	2.7	3.9	7.7	15.9	65.0
Camden	4.4	0.7	4.6	5.9	4.9	4.5	4.0
Erie	5.3‡	1.5	0.8	2.3	6.9	49.0	46.6

* Population figures furnished by the health department: 115,333.
† Data for 1925 only.
‡ Population figures furnished by the health department: 133,000.

TABLE 3

Deaths in cities in South Atlantic States from typhoid per hundred thousand population

	1928	1927	1926	1921–1925	1916–1920	1911–1915	1906–1910
Norfolk	1.6	1.1	1.7	2.8	8.8	21.7	42.1
Jacksonville	2.1	7.2	8.8				
Washington	2.7	1.8	2.5	5.4	9.5	17.2	36.7
Richmond	3.1	0.0	2.6	5.7	9.7	15.7	34.0
Baltimore	3.8	1.8	4.7	4.0	11.8	23.7	35.1
Wilmington	4.7	3.2	2.4	4.7	25.8*	23.2†	33.0
Atlanta	7.4	14.0	17.2	14.5	14.2	31.4	58.4

* 1916 only.
† Lacks 1913.

1925. For the first time in recent years, the rates of New York and Pittsburgh show slight increases.

The 1928 typhoid death rate in the Middle Atlantic cities as a whole (table 12) is a trifle higher (1.50) than the 1927 rate (1.41), representing 177 deaths as against 166 in the preceding year.

While the 1928 total typhoid death rate (3.72) for the South Atlantic cities as a group (table 12) is somewhat higher than the 1927 rate (3.39), the highest rate in the group (table 3) in 1928 is decidedly lower than in 1927 or any preceding year. Atlanta, for the first time

TABLE 4

Deaths in cities in East North Central States from typhoid per hundred thousand population

	1928	1927	1926	1921–1925	1916–1920	1911–1915	1909–1910
Youngstown	0.0	2.4	0.0	7.2	19.2	29.5	35.1
Akron	0.4*	2.8	1.5	2.4	10.6	21.0	27.7†
Chicago	0.5	0.7	0.8	1.4	2.4	8.2	15.8
Cleveland	0.6	1.0	1.4	2.0	4.0	10.0	15.7
Grand Rapids	0.6	1.9	1.3	1.9	9.1	25.5	29.7
Milwaukee	0.7	0.9	1.7	1.6	6.5	13.6	27.0
Canton	0.8	0.0	3.6	3.3	8.9		
Detroit	1.0	1.2	2.2	4.1	8.1	15.4	22.8
Cincinnati	1.7	3.9	2.7	3.2	3.4	7.8	30.1
Columbus	1.7	2.0	1.7	3.5	7.1	15.8	40.0
Dayton	2.7	1.7	2.3	3.3	9.3	14.8	22.5
Flint	3.4	2.1	0.7	4.6	22.7	18.8	46.9
Indianapolis	3.6	1.3	5.4	4.6	10.3	20.5	30.4
Toledo	4.8	2.9	4.7	5.8	10.6	31.4	37.5

* Population figures furnished by the health department: 225,000.
† Data for 1909 and 1910 only.

since these summaries were begun with the single exception of 1919, when its rate was 9.6, has a typhoid rate below 10.0; its 1928 rate (7.4) is about half of its 1927 rate (14.0). It is still, however, considerably higher than the other rates in the South Atlantic group; Atlanta is the only city of the group to fall below first rank in 1928 (table 9). Jacksonville, which had 1926 and 1927 rates of 8.8 and 7.2, respectively, has in 1928 the second lowest rate in the group (2.1). Atlanta and Jacksonville are the only cities in the group with lower rates in 1928 than in 1927. The increases in the other five cities,

however, are not important, the largest being that of Richmond, which in 1927 had no typhoid deaths and in 1928 had six, giving a rate of 3.1. Norfolk has regained its customary place at the head of the table (last year yielded to Richmond), but with a rate slightly higher than in 1927. Although Baltimore and Washington in 1927 had about the same rates as Boston and New York, in 1928 the Baltimore and Washington rates of 3.8 and 2.7, respectively, are at least double those of Boston and New York (0.6 and 1.4). Though Baltimore's 1928 rate is more than twice its own 1927 rate (3.8 as against 1.8), the Maryland Department of Health, in the report of its Bureau of Sanitary Engineering for 1928, states that "the year 1928 shows by far the lowest typhoid fever incidence in the history of the state. The typhoid and paratyphoid fever mortality for the past year was 5.2 per 100,000 for the state at large. . . . . Ten years ago

TABLE 5

Deaths in cities in East South Central States from typhoid per hundred thousand population

	1928	1927	1926	1921–1925	1916–1920	1911–1915	1909–1910
Louisville	2.7	3.1	6.7	4.9	9.7	19.7	52.7
Birmingham	7.2	12.9	8.5	10.8	31.5	41.3	41.7
Memphis	11.6	14.5	19.2	18.9	27.7	42.5	35.3
Nashville	15.0	16.0	35.0	17.8	20.7	40.2	61.2

the rate for Maryland was 16.9." Of the seven cities in this group, five have rates between 2.0 and 5.0 and are therefore in the first rank (table 9), one (Norfolk) is on the honor roll and one (Atlanta) in the second rank.

As in previous years, in 1928 the South Atlantic group average (3.72) is decidedly lower than the group averages for the other southern groups (West South Central cities, 6.18; East South Central cities, 7.68).

The total typhoid death rate (table 12) in the group of East North Central cities in 1928 is a little lower than in 1927 (1.10 as against 1.31) and lower than in 1925 or 1926 (2.19, 1.69). Of the fourteen cities in the group, nine have lower rates in 1928 than in 1927 (table 4), and four of the nine have had continuous declines beginning in 1926 (one, Detroit, has had a steady decrease since 1920). Youngs-

town's rate of 0.0 makes the third such rate in successive years in this group, Youngstown having attained it also in 1926 and Canton in 1927. The highest 1928 rate (Toledo, 4.8), however, is higher than the corresponding 1927 rate (Cincinnati, 3.9), and there are three cities with 1928 rates of over 3.0, whereas there was only one such rate in 1927. Toledo, at the foot of this table in 1928, has been either at the foot or next to the foot every year since 1924, when these summaries first grouped the cities geographically. Chicago in 1928 lowered its excellent 1927 rate of 0.7 with a rate of 0.5, this being its third successive year with a rate below 1.0. Cleveland and Detroit,

TABLE 6

Deaths in cities in West North Central States from typhoid per hundred thousand population

	1928	1927	1926	1921–1925	1916–1920	1911–1915	1906–1910
Duluth	0.0	2.7	0.8	1.7	4.4	19.8	45.5
Minneapolis	0.2	0.7	1.4	1.9	5.0	10.6	32.1
Des Moines	1.3	3.3	2.7	2.2	6.4	15.9	23.7
St. Paul	1.3*	2.8	1.2	3.4	3.1	9.2	12.8
Kansas City, Kan	1.7	0.0	4.3	5.0	9.4	31.1	74.5†
St. Louis	2.2	1.9	2.3	3.9	6.5	12.1	14.7
Kansas City, Mo	2.3	2.9	3.5	5.7	10.6	16.2	35.6
Omaha	2.7	0.9	1.8	3.3	5.7	14.9	40.7

* Population figures furnished by the health department: 298,000.

† Lacks data for 1906 and 1907.

both with very low rates in 1927 (1.0 and 1.2), have still lower rates in 1928 (0.6 and 1.0).

The East South Central cities as a group (table 12) have a much lower typhoid death rate in 1928 (7.68) than 1927 (10.07). It is still, however, the highest group rate in the country. Every one of the four cities has a lower rate in 1928 than in 1927, the 1928 range being 2.7–15.0 as against a 1927 range of 3.1–16.0. The order of the cities' rates is the same as usual, Louisville standing at the head of the table with rates well below the others, and Birmingham holding second place. The reductions in Birmingham and Memphis are especially notable, Birmingham's rate declining from 12.9 in 1927 to 7.2 in 1928, and Memphis' 1927 rate being 14.5 and its 1928 rate 11.6. None of these cities are on the honor roll (table 9); one is in the first

rank, one in the second and two in the third. The Memphis and Nashville rates are the highest in the country, as they were also in 1924, 1926 and 1927.

Among the West North Central cities (table 6) in 1928 there is one city (Duluth) with no typhoid deaths, just as there was in 1927 (Kansas City, Kans.). While the 1927 range of rates was 0.0–3.3, the 1928 range is 0.0–2.7, the lowest range in any group. Five of the eight cities have in 1928 rates lower than in 1927, and the average for the group as a whole (table 12) is a little lower (1.65 as against 1.86). The decreases and increases in all instances are quite small.

TABLE 7

Deaths in cities in West South Central States from typhoid per hundred thousand population

	1928	1927	1926	1921–1925	1916–1920	1911–1915	1906–1910
San Antonio	2.3	5.7	7.8	9.3	23.3	29.5	35.9
Tulsa	3.5	8.7	14.3	16.2*			
Dallas	5.0	6.6	8.9	11.2	17.2		
Houston	6.0†	5.6	6.0	7.6	14.2	38.1	49.5‡
Fort Worth	7.0	4.3	10.7	6.1	16.3§	11.9	27.8
New Orleans	7.2	8.0	18.6	11.6	17.5	20.9	35.6
Oklahoma City	10.0¶	6.4					
El Paso	10.2	7.9	9.2	10.8	30.7	42.8	

* Lacks data for 1921 and 1922.

† Population figures furnished by the health department: 300,000.

‡ Data for 1910 only.

§ Lacks data for 1918 and 1919.

¶ Rate calculated from population figures for 1927: 140,000.

Only two cities have had continuously declining rates, beginning with 1926.

In the West South Central group (table 7), four of the eight cities had lower rates in 1928 than in 1927, substantial improvement appearing in San Antonio (a decrease from 5.7 to 2.3) and especially in Tulsa with a 1928 rate of 3.5 as against a 1927 rate of 8.7; Tulsa's 1928 rate is only a fourth of its 1926 rate (14.3). On the other hand, whereas all the cities in this group in 1927 had rates under 9.0, there are two rates over 9.0 in 1928 (Oklahoma City and El Paso). In 1927, seven of the cities fell into the second rank (table 9), the eighth into first; in 1928, two are in the first rank, four are in the second, and

two are in the third. The group rate (table 12) for these West South Central cities in 1928 (6.18) is a little lower than the 1927 rate (6.71) both being far below the rates in 1926 and 1927 (13.27 and 11.69), both being also next to the highest group rates in the country.

Tacoma, which stood at the foot of the table of the Mountain and Pacific cities (table 8) in 1927 and next to the foot in 1926, heads the list in 1928 with a record of no typhoid deaths, the second city in this group ever to attain this distinction (the other was Spokane in 1919). Moreover, Tacoma in 1928 had no diphtheria deaths; no other city of these summaries has ever reported a clear record for both of these diseases in the same year. Five of the ten cities have somewhat

TABLE 8

Deaths in cities in Mountain and Pacific States from typhoid per hundred thousand population

	1928	1927	1926	1921–1925	1916–1920	1911–1915	1906–1910
Tacoma	0.0	2.8	3.8	3.7	2.9	10.4	19.0
Oakland	1.1	2.2	0.8	2.0	3.8	8.7	21.5
Seattle	1.3	2.7	2.3	2.6	2.9	5.7	25.2
Los Angeles	1.5*	1.0	1.1	3.0	3.6	10.7	19.0
San Diego	1.7	0.9	0.9	1.6	7.9	17.0	10.8
Spokane	1.8	0.9	6.5	4.4	4.9	17.1	50.3
Portland	2.1†	2.1	2.5	3.5	4.5	10.8	23.2
Denver	2.7	2.8	3.5	5.1	5.8	12.0	37.5
Salt Lake City	2.9	2.2	0.7	6.0	9.3	13.2	41.1
San Francisco	3.4	1.7	2.5	2.8	4.6	13.6	27.3

* Population figures furnished by the health department: 1,400,000.
† Population figures furnished by the health department: 336,163.

higher rates in 1928 than in 1927, and two of the 1928 rates are higher than any of the 1927 rates. The rate for the group as a whole (table 12) is a trifle higher in 1928 (1.92) than in 1927 (1.74).

Nine of the eighty-one cities concerned in this study (table 9) had no typhoid deaths during 1928 (there were seven such cities in 1927). Four of the nine have had similar records once before, and the fifth (Yonkers) four times before. Four of them are New England cities and two belong to the Middle Atlantic group. Every geographic division is represented among them except the three Southern groups.

Of the forty-five cities on the honor roll (rates below 2.0) in 1928, thirty were there also in 1927. The six cities with populations of

TABLE 9

Death rates from typhoid in 1928

Honor roll (from 0.0 to 1.9 deaths per hundred thousand)			
Duluth	0.0	**Philadelphia**	0.8
Elizabeth	0.0	**Rochester**	0.9
Lowell	0.0	**Detroit**	1.0
Lynn	0.0	**Newark**	1.0
New Haven	0.0	**Oakland**	1.1
Springfield, Mass	0.0	**Hartford**	1.2
Tacoma	0.0	**Des Moines**	1.3
Yonkers	0.0	**St. Paul**	1.3
Youngstown	0.0	**Seattle**	1.3
Minneapolis	0.2	**New York**	1.4
Akron	0.4	**Paterson**	1.4
Chicago	0.5	**Scranton**	1.4
Syracuse	0.5	**Los Angeles**	1.5
Worcester	0.5	**Cambridge**	1.6
Boston	0.6	**Norfolk**	1.6
Bridgeport	0.6	**Cincinnati**	1.7
Cleveland	0.6	**Columbus**	1.7
Grand Rapids	0.6	**Kansas City, Kan**	1.7
Jersey City	0.6	**Providence**	1.7
Milwaukee	0.7	**San Diego**	1.7
Albany	0.8	**Spokane**	1.8
Canton	0.8	**Utica**	1.9
New Bedford	0.8		
First rank (from 2.0 to 4.9)			
Jacksonville	2.1	**Salt Lake City**	2.9
Portland, Ore	2.1	**Richmond**	3.1
Trenton	2.1	**Flint**	3.4
St. Louis	2.2	**Pittsburgh**	3.4
Buffalo	2.3	**San Francisco**	3.4
Kansas City, Mo	2.3	**Tulsa**	3.5
San Antonio	2.3	**Indianapolis**	3.6
Reading, Pa	2.6	**Baltimore**	3.8
Dayton	2.7	**Camden**	4.4
Denver	2.7	**Fall River**	4.5
Louisville	2.7	**Wilmington**	4.7
Omaha	2.7	**Toledo**	4.8
Washington	2.7		
Second rank (from 5.0 to 9.9)			
Dallas	5.0	**Birmingham**	7.2
Erie	5.3	**New Orleans**	7.2
Houston	6.0	**Atlanta**	7.4
Fort Worth	7.0		
Third rank (above 10.0)			
Oklahoma City	10.0	**Memphis**	11.6
El Paso	10.2	**Nashville**	15.0

TABLE 10

Number of cities with various typhoid death rates

	NUMBER OF CITIES	10.0 AND OVER	5.0 TO 9.9	2.0 TO 4.9	1.0 TO 1.9	0.1 TO 0.9	0.0
1906–1910	74	72	2	0	0	0	0
1911–1915	75	55	18	2	0	0	0
1916–1920	77	19	29	29	0	0	0
1921–1925	79	8	15	46	10	0	0
1926	80	6	10	27	19	14	4
1927	81	4	8	26	23	13	7
1928	81	4	7	25	20	16	9

TABLE 11

*Total typhoid rate for seventy-four cities, 1910–1928**

	POPULATION	TYPHOID DEATHS	TYPHOID DEATHS PER 100,000
1910	22,286,000	4,586	20.58
1911	22,916,700	3,883	16.94
1912	23,535,450	3,077	13.07
1913	24,151,936	3,222	13.34
1914	24,776,777	2,744	11.07
1915	25,392,422	2,373	9.34
1916	25,928,745	2,154	8.31
1917	26,528,213	1,963	7.40
1918	26,737,190†	1,796†	6.72
1919	27,373,579†	1,141†	4.17
1920	28,182,528	1,074	3.81
1921	28,509,732	1,130	3.96
1922	28,947,007	955	3.30
1923	29,580,000	936	3.16
1924	30,155,014	937	3.11
1925	30,938,501	1,067	3.45
1926	31,667,424	895	2.83
1927	32,492,123	638	1.96
1928	33,116,784	627	1.89

*** The following seven cities are omitted because data for the full period are not available: Canton, Dallas, Jacksonville, Oklahoma City, Tulsa, Utica, Wilmington.**

† Lacks data for Fort Worth.

more than a million (Chicago, Cleveland, Detroit, Los Angeles, New York City, Philadelphia) are all on the honor roll. Four geographic divisions (New England, East North Central, West North

Central, Mountain and Pacific) have all their cities on the honor roll or in the first rank. Erie is the only Northern city among the eleven cities comprising the second and third ranks, the Southern groups being represented as follows: West South Central, six times; East South Central, three times; South Atlantic, once. Erie and Fort Worth are the only cities in the second and third ranks in 1928 which were not in them in 1927.

Twenty-five of the cities have rates below 1.0, as against twenty in 1927 (table 10). There are in 1928 only four cities of the eighty-one

TABLE 12

Total typhoid deaths per hundred thousand population in eighty-one cities according to geographic divisions

	POPULATION	1928		1927		1926	1925
		Typhoid deaths	Typhoid death rate	Typhoid deaths	Typhoid death rate	Typhoid death rate	Typhoid death rate
New England	2,548,088	23	0.90	32	1.26	1.51	2.37
Middle Atlantic	11,790,733	177	1.50	166	1.41	2.09	2.97
South Atlantic	2,285,300	85	3.72	76	3.39	5.38	5.71*
East North Central	8,512,300	94	1.10	109	1.31	1.69	2.19
East South Central	881,600	68	7.68	86	10.07	14.47	14.30
West North Central	2,602,800	43	1.65	47	1.86	2.22	3.31
West South Central	1,764,200	109	6.18	114	6.71	11.69†	13.27†
Mountain and Pacific	3,750,263	72	1.92	61	1.74	1.98	2.19

* Lacks data for Jacksonville.

† Lacks data for Oklahoma City.

with rates over 10.0, whereas the 1906–1910 averages of seventy-two of seventy-four cities were over 10.0.

The total typhoid death rate for seventy-four cities in 1928 (table 11) is a little lower than in 1927 (1.89 as against 1.96). There were in these seventy-four cities during 1928 a total of 627 typhoid deaths. If the typhoid death rate of 1910 (20.58) had prevailed in these cities in 1928, there would have been 6,815 deaths, almost eleven times the number that actually occurred.

The different groups of cities (table 12) if ranked according to their 1928 rates would be in about the same order as in all the years beginning with 1925: the New England, East North Central, Middle Atlantic, West North Central and Mountain cities all have low group

rates very nearly alike (in 1928 all are below 2.0), while the three Southern groups rank in the following order, as they have every year beginning with 1925: South Atlantic, West South Central, East South Central. The rates of the two latter (6.18 and 7.68) are considerably higher, in 1928 as in the other years, than the South Atlantic rate (3.72). The 1928 rates in the three Southern groups are far below their rates for 1925 and 1926, and, in the case of the East South Central cities, far below the 1927 rate as well. The progressive decline in the rates of seven of the eight groups for the three years 1925–1927 has been broken in 1928 in three instances (Middle Atlantic, South Atlantic, Mountain and Pacific). The increases, however, are small, the largest being that of the South Atlantic group from 3.39 to 3.72.

DISCUSSION

BOILER WATER CONDITIONING

We should like to make a somewhat belated contribution to the discussion on the paper of Dr. R. E. Hall and his associates on boiler-water conditioning which appeared in the January number.[1] We should the more appreciate this opportunity as our paper on priming which appeared in the *Journal of the Society of Chemical Industry* (1927, p. 315, T) has not yet been abstracted in this JOURNAL.

Dr. Hall refers on page 86 to the fact that his conclusions in part agree and in part disagree with those reached by other workers, including ourselves, and we think it desirable to draw attention to three points in particular in which there are outstanding differences between us.

(*a*) All Dr. Hall's results on priming or foaming are based on experiments carried out at atmospheric pressures. We consider that the nature of the ebullition in a boiler containing salt solutions with or without suspended solids cannot be inferred from atmospheric pressure experiments.

(*b*) At 150 pounds pressure, suspended solids have no effect in promoting or increasing priming in boiler waters carrying a concentration of dissolved solids up to 10,000 p.p.m. Suspended solids do, however, act as promoters or increasers of priming if the boiler is operated at 40 pounds pressure. It is not clear as to whether Dr. Hall considers that his conclusions based on atmospheric pressure experiments can be applied to practical boiler conditions; we ourselves are convinced that they are inapplicable.

(*c*) The influence of calcium compounds as regards priming is not appreciably different from that of sodium compounds.

(*d*) Sodium carbonate has no greater effect in promoting priming than other inorganic salts. This statement might require modification if organic matter is present in water.

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[1] JOURNAL, January, 1929, page 79.

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ABSTRACTS OF WATER WORKS LITERATURE[1]

FRANK HANNAN

Key: American Journal of Public Health, 12: 1, 16, January, 1922. The figure 12 refers to the volume, 1 to the number of the issue, and 16 to the page of the Journal.

New Pumping and Filtration Plant for Knoxville Water Works. CHAS. B. BURDICK. Eng. News-Rec., 101: 204–7, August 9, 1928. Illustrated description of new Williams Creek plant, treating Tennessee River water. Population is estimated at 100,000 and consumption is 9 million gallons per day. All services have been metered since 1917. Intake, about 700 feet upstream where low water depth is 10 to 15 feet 80 feet from shore, consists of 2 lines of 36-inch pipe having outer ends supported upon piers of precast reinforced concrete. Water enters through 1-inch holes spaced 2 inches center to center in outer 3 lengths. Ends of pipes are closed with perforated plates. The circular mechanical mixing chambers, of which there are 2, each providing ten minutes' retention, can be operated in parallel or series. Two coagulation basins provide six-hour coagulation period at 15-million gallon per day rate. Location made possible installation of hopper bottoms, from which practically all sediment can be discharged by gravity through 12-inch cast iron pipes without emptying basins. There are six 2.5-million gallon per day rapid sand filters, with 0.84-million gallon clear well below. Alum and occasionally lime are employed in treatment. For high-lift service there are 2 turbo-centrifugal pumping units, each consisting of multi-stage steam turbine geared to shaft which drives 2 centrifugal pumps in series, delivering 15-million gallons per day against 350-foot head. Extension of shaft is connected directly to 300-kilowatt, direct-current generator for driving one low-lift pump and miscellaneous station motors. Low-lift plant consists of 2 centrifugal pumps of 18-million gallon per day capacity against 55-foot head driven by 220-horsepower motors. Under heads of 53 to 60 feet these units developed 78.3 to 79.4 per cent over-all efficiency, indicating pump efficiencies of about 85 per cent. There are two 500-horsepower water-tube boilers, for 300 pounds pressure with 200° of superheat. On test, at slightly above rating, boilers and stokers developed 80.4 and 83.2 per cent efficiency, using coal of 13,800 British thermal units. Other improvements include elevated reinforced concrete reservoirs of 10- and 1-million gallons capacity.—*R. E. Thompson.*

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[1] Vacancies on the abstracting staff occur from time to time. Members desirous of coöperating in this work are earnestly requested to communicate with the chief abstractor, Frank Hannan, 285 Willow Avenue, Toronto 8, Ontario, Canada.

Spring Water Gravity Supply for Ada, Okla. N. T. Veatch, Jr. Eng. News-Rec., 101: 236–7, August 16, 1928. Recent improvements in water supply of Ada, derived from large spring 12 miles from city, are described. Concrete curb wall around spring and adjacent area form basin from which water flows 4000 feet to pump house where it is pumped by hydraulic-turbine-driven centrifugal pumps to concrete reservoir near city, and then pumped again into distribution system. Chief improvement will be new-7-million gallon per day, 24-inch cast iron pipe line which will convey water by gravity to city reservoir. When subjected to pressure test, hydrostatic test heads will be 75, 130, 180 and 235 feet for working pressure heads of 50, 100, 150 and 200 feet respectively. It is specified that leakage shall not exceed 200 gallons per day per inch-mile under maximum hydraulic grade pressure maintained for forty-eight hours. Population of city is 9000 and well flow is somewhat in excess of 7 million gallons.—*R. E. Thompson.*

Foundry Inspection of Pipe Pays. W. C. Hawley. Eng. News-Rec., 101: 243, August 16, 1928. Practice of requiring all cast iron pipe to be inspected at foundry before being coated is becoming quite general. Prior to 1902 pipe was purchased by Pennsylvania Water Company, Wilkinsburg, Pa., without inspection. Since that time all pipe has been inspected. Possibly two-thirds of the 210 miles of pipe now in system was inspected when purchased. Practically all breaks in recent years have been in pipe purchased without inspection and have been due to defects detectable by inspection. Damage to property has amounted to many times the cost of inspection. Only breaks in inspected pipe have been square breaks in 4- or 6-inch diameters due probably either to heavy traffic or poor foundation conditions.—*R. E. Thompson.*

Elimination of Iron from Textile Waters. B. M. Conaty. Textile Colorist 49: 619–20, 1927. From Chem. Abst., 22: 873, March 10, 1928.—*R. E. Thompson.*

Preservation of Starch Solution. Naotsuna Kanô. Sci. Repts. Tôhoku Imp. Univ., 16: 861–3, 1927. From Chem. Abst., 22: 741, March 10, 1928. For use in acid solutions, addition of 0.5 cc. of 2 N hydrocholoric acid to 50 cc. starch solution proved most satisfactory. For titrations in neutral solutions, addition of few drops of carbon bisulfide to 50 cc. starch is recommended. After standing 8 months scarcely any turbidity was noticed in starch solutions containing either hydrochloric acid or carbon bisulfide.—*R. E. Thompson.*

Determination of Very Small Quantities of Iodine. P. A. Meerburg. Z. physik. Chem., 130: 105–8, 1927. From Chem. Abst., 22: 741, March 10, 1928. In studying theory that goiter is due to iodine-deficiency, accurate determinations of iodine in drinking water are necessary. Method of v. Fellenberg (C. A., 18: 2192) of distinguishing between iodine in organic and in inorganic combination has not proved entirely satisfactory and following modified procedure has been developed for determination of total iodine content of water. To 3 to 6 liters of water add few drops phenolphthalein indicator solution and few cubic centimeters 0.6 per cent potassium carbonate solution. Evaporate

to 150 cc., keeping solution alkaline, and filter off calcium carbonate, iron hydroxide, etc. Evaporate filtrate in platinum dish, and digest residue of moist salts with 3 portions of 80 to 95 per cent alcohol. Evaporate residue to dryness, ignite carefully and again digest in alcohol 3 times. Combine alcohoiic extracts, evaporate to dryness, take up in 1 cc. water and treat with potassium nitrite, sulfuric acid and chloroform, comparing color of chloroform with that obtained similarly with known amounts of iodine.—*R. E. Thompson.*

Apparatus for Deaërating Water or Other Liquids. R. C. Jones. U. S. 1,654,260–1–2, December 27, 1927. From Chem. Abst., 22: 702, March 10, 1928.—*R. E. Thompson.*

Strain Effects in Mild Steel. Henry S. Rawdon. Eng. News-Rec., 101: 244–50, August 16, 1928. An extensive discussion of strain in mild steel, its effects and detection. The corrodibility may be greatly increased as result of permanent strain. Strained metal behaves as anode with respect to unstrained metal and is therefore corroded. Example cited illustrating how corrosion of boiler tube was localized where tube had been deformed by hammering, attack being so severe that perforation resulted within few months.—*R. E. Thompson.*

Building a Brick-Faced Concrete-Arch Dam. Herbert W. Reutershan. Eng. News-Rec., 101: 268–72, August 23, 1928. Illustrated description of Caneadea dam, which is concrete arch faced on both sides with vitrified brick to resist frost action and to prevent disintegration of concrete. Dam is located on Caneadea Creek at junction with Genesee River in narrow gorge 300 feet deep, and will impound water for Rochester Gas and Electric Corporation power plants. Lake formed will be 2 miles long and 1½ miles wide, covering 800 acres and impounding 1,200,000,000 cubic feet. Structure is constant-angle arch, 140 feet above stream bed, with maximum radius of 262 feet and total arch length of 440 feet. Abutments are of gravity section.—*R. E. Thompson.*

Studies for Additional Water Supply for Milwaukee, Wis. C. S. Gruetzmacher. Eng. News-Rec., 101: 273, August 23, 1928. Brief data given on comprehensive study being carried out in connection with additional water supply for Milwaukee. Water consumption has increased from 66.9 million gallons per day in 1920 to 76.5 in 1927, and maximum daily consumption from 89 to 106.5, with peak rate in 1927 of 190 million gallons. Water quality survey is being conducted to determine best location for new intake in Lake Michigan.—*R. E. Thompson.*

Boyds Corners Dam—Why it was Declared Unsafe. Eng. News-Rec., 101: 250–1, August 16, 1928. Additional data.—*R. E. Thompson.*

Corrosion of Copper and Brass. K. Inamura. Science Repts. Tôhoku Imp. Univ., 16: 999–1008, 1927. From Chem. Abst., 22: 753, March 10, 1928. Studies were made of corrosion of copper and 60–40 and 70–30 brasses in various dilute acids, bases, and salt solutions. Tests were made by suspending test

pieces in flasks of corroding medium, unagitated, and measuring loss in weight. —*R. E. Thompson.*

Action of Water, Air, and Carbon Dioxide on the Corrosion of Iron. K. INAMURA. Science Repts. Tôhoku Imp. Univ., 16: 979–86, 1927. From Abst., 22: 753, March 10, 1928. Investigation of influence of oxygen and carbon dioxide on corrosion of iron in water. It is shown that corrosion of iron depends on presence of oxygen in water free from carbon dioxide and that carbon dioxide accelerates corrosion in presence of oxygen. Corrosion may proceed in water containing carbon dioxide in absence of oxygen.—*R. E. Thompson.*

Testing of Protective Coatings. E. A. OLLARD. Metal Ind., (London), 31: 385–7, 416–8, 1927. From Chem. Abst., 22: 753, March 10, 1928. Apparatus described for testing resistance to corrosion by liquid, liquid being moved periodically to change water level and produce some erosion. Suspended particles should be present in liquid, and temperature should be controlled. —*R. E. Thompson.*

Protection Against Corrosion by Means of Metallic Coatings. WM. BLUM. J. Chem. Education 4: 1477–87, 1927. From Chem. Abst., 22: 753, March 10, 1928.—*R. E. Thompson.*

Installation and Upkeep of Pipelines. P. WIEGLEB. Chem.-Ztg., 51: 881–3, 903–4, 1927. From Chem. Abst., 22: 892, March 20, 1928. Review.—*R. E. Thompson.*

The Molecular Structure of Water. H. M. CHADWELL. Chem. Reviews, 4: 375–98, 1927. From Chem. Abst., 22: 895, March 20, 1928. Evidence of association of molecules reviewed. Effect of dissolved substances on polymerization is discussed and relation of problem to study of aqueous solutions in emphasized. About 100 references included.—*R. E. Thompson.*

Corrosion-Resisting Steels. L. PERSOZ. Rev. chim. ind., 36: 397–400, 1927. From Chem. Abst., 22: 937, March 20, 1928. Compositions given of stainless steels for various purposes.—*R. E. Thompson.*

The Effect of Impurities on the Corrosion of Metals. C. J. SMITHELLS. World Power, 9: 85–93, 1928; of C. A. 21: 2246. From Chem. Abst., 22: 938, March 20, 1928. Survey of more recent findings.—*R. E. Thompson.*

Removal of Manganese from Water by Means of Natural and Artificial Black Sands. LÜHRIG. Gas u. Wasserfach, 70: 1277–81, 1927. From Chem. Abst., 22: 1000, March 20, 1928. Capacity of black sands, both artificial and natural, for removing manganese from water is increased by access of oxygen, or use of oxidizing agents. Periodic washing with potassium permanganate has been used, but other oxidizing agents such as per-salts, hypochlorites, etc., also serve, although presence of free chlorine is detrimental. No biological process is involved in mechanism of action of black sand.—*R. E. Thompson.*

The Removal of Silica from Waters Containing Silicic Acid. E. BERL and H. STAUDINGER. Z. angew. Chem., **40**: 1313–7; Z. Ver. deut. Ing., **71**: 1654–7, 1927. From Chem. Abst., **22**: 1000, March 20, 1928. Solubility of calcium silicate is increased by presence of sulfates and chlorides, probably through formation of compounds of type $NaHCaSiO_4$. Effect of these ions is proportionate to relative concentrations. Calcium hydrate in double calculated quantity will satisfactorily remove silicates in presence of chlorides in ratios up to silica: sodium chloride equals 1925. Colloidal silica is coagulated by treatment with alum.—*R. E. Thompson.*

The Use of Colloids to Prevent Boiler Scale. E. SAUER and F. FISCHLER. Z. angew Chem., **40**: 1176–83, 1276–9, 1927. From Chem. Abst., **22**: 1000, March 20, 1928. Study of precipitation of calcium carbonate from water showed that stirring and higher temperatures increase degree of removal proportionately. Water containing hydrosols of gelatin, gum arabic, agar, dextrin, and tannin shows greater hardness after treatment than water free of these substances. Concluded that these materials are protective colloids for carbonates present, and therefore interfere with softening process.—*R. E. Thompson.*

Boiler Scale and Its Prevention. HERMANN WALDE. Wiss. Veröffentlich. Siemens Konzern, **6**: 151–70, 1927. From Chem. Abst., **22**: 1000, March 20, 1928. Ratios: calcium sulfate/calcium carbonate, silica/calcium oxide, magnesium oxide/calcium oxide, as determined by analysis, give indication as to nature of boiler scale which will be produced. Three kinds of scales are recognized, namely: hard stones, adherent scales, and mud. Chemical, physical and electrical methods of preventing boiler scale are discussed. Action of Cumberland electrolytic method is explained.—*R. E. Thompson.*

Hydrosan. AUGUST HUMMEL. Textile Colorist, **49**: 675–7, 1927. From Chem. Abst., **22**: 1000, March 20, 1928. Hydrosan is added to hard water in proportion of 35 grams per 1000 liters per degree of hardness to prevent the formation of or to remove calcium soaps. It should be added to water either before or with soap, with which it may be incorporated. It is claimed to retard hydrolysis of soap and save both soap and alkali. A satisfactory lather can be obtained in sea water with this substance. As it is placed directly in water with the soap, no special apparatus is required to treat the water.—*R. E. Thompson.*

Arch Dam of Unusual Design. Eng. News-Rec., **101**: 311, August 30, 1928. Novel type of arch dam has been developed by two Frenchmen, MESNAGER and VEYRIE. Principle is that, as cost of dam increases rapidly with height, it is cheaper to build series of low dams than one high one. First arch is built to full height, but is of unusually light construction. Immediately below this is second light arch of lesser height, below this a third still lower, and so on for as many steps as necessary. Light construction is made possible by intermediate pools, which provide balanced load on lower downstream section of each arch thereby materially lowering effective water pressure against each dam. Not

only does this reduce amount of material in structure but it also prevents scour below dam at times of overflow. Another advantage is that after construction is completed, dam can be thoroughly tested by subjecting it to double, triple or even greater multiples of usual working pressure, simply by emptying one or more of intermediate pools. This can be repeated at any time. Failure of any wall would not be followed by complete loss of structure, but would simply double pressure on adjacent wall. Tests were made with plastic models and mercury. Safety factor of design was found to be between 4 and 5. Government subsidy of 200,000 francs has been granted to inventors for further experimentation, and plans have been prepared for dam of this type on Upper Dordogne River.—*R. E. Thompson.*

Change of Water Rights Doctrine in State of Washington. Eng. News-Rec., 101: 317, August 30, 1928. In PROCTOR *vs.* SIM, 134 Wash. 606, Supreme Court (of Washington) expressly repudiates ancient doctrine that riparian owners are entitled to "have the waters lap their shores as they were by nature wont to do, undisturbed and undiminished," etc., and reaffirms doctrine laid down in BROWN *vs.* CHASE, 125 Wash. 542, to effect that "waters of non-navigable streams in excess of amount which can be beneficially used, either directly or prospectively, within a reasonable time, or in connection with riparian land, are subject to appropriation for use on non-riparian lands." During last decade there has been gradual change to acceptance of this beneficial-use doctrine in State of Washington.—*R. E. Thompson.*

Water Supply Struggles of a Small Southern City. PRESON P. PHILLIPS. Eng. News-Rec., 101: 361–3, 1928. The recently completed water works of Mount Airy, N. C., treating water drawn from Lovill's Creek, consists of a mixing chamber, 2 coagulation basins, three 0.5-million gallon rapid sand filters and a 0.5-million gallon clear well. The plant, which is of 1.5 million gallons capacity, will provide 200 gallons per capita daily to the present population of 7500. The mixing basin, which is of the over-and-under baffled type, provides a retention period of fifteen minutes, and the 2 coagulation basins of five hours. The cost of the plant was approximately $140,000.—*R. E. Thompson (Courtesy Chem. Abst.).*

Full Storage Demanded Before Universal Metering at Colorado Springs. FRANK O. RAY. Eng. News-Rec., 101: 280–1, August 23, 1928. Colorado Springs is supplied by gravity from limited mountainous drainage area adjacent to city. Area cannot be extended without encroaching on watersheds of other municipalities. All water in Colorado is owned either through prior use or purchase. Plan for development of water system proposed in 1916 provides for construction of number of reservoirs to store practically all runoff. Land has been secured, filings on the water have been made and construction is in progress. Practically all the water supply and storage is obtained at elevation of 9000 feet or more above sea level, in mountainous district with steep slopes, and costs about $300 per acre-foot or $900 per million gallons. City owns both water and electric systems. Hydro plants are located on water supply lines and pay water department for use of water.

There is a high-pressure distribution system operating at 165 pounds pressure to which the low-pressure system at about 45 pounds is connected by means of pressure-regulating valves. Water for lawns and parks is furnished from direct stream flow through irrigation canals and low-pressure pipe lines. Large consumers are metered, there being 400 services metered out of total of 13,000. Storage development should be completed and system then completely metered.—*R. E. Thompson.*

Coefficients of Venturi Meters and Reynolds' Criterion. W. S. PARDOE. Eng. News-Rec., 101: 281–2, August 23, 1928. Writer believes that coefficients of Venturi meters plotted against REYNOLDS' criterion cannot give any satisfactory result above critical velocity. Data in support of this contention are presented. All experimental evidence available to writer indicates that coefficient increases with diameter and does not vary greatly with proportional roughness or viscosity. That is, there is loss depending on form of meter which is greater than computed frictional loss (see Eng. News-Rec., September 25, 1919) and is independent of surface roughness. Hence to obtain coefficient of large Venturi meters we must still use series of 2 or 3 model meters inside the capacity of our laboratories and extrapolate. Errors of extrapolation cannot be very great. Both theory and experiment indicate that REYNOLDS' criterion can be used advantageously below critical velocity, or for stream-line flow.—*R. E. Thompson.*

Some Theoretical and Practical Data on Javel Water and on Bleaching Powder. J. H. FRYDLENDER. Rev. prod. chim., 30: 881–4, 921–7, 1927. From Chem. Abst., 22: 1018, March 20, 1928. Review of preparation and properties.—*R. E. Thompson.*

Extremely Thin Dam Fails. Eng. News-Rec., 101: 318, August 30, 1928 Brief illustrated description of partial failure of small concrete dam of Union Ice Company on Prosser Creek. Dam was of unreinforced concrete, 35 feet high and 100 feet in length, thickness at top being only 18 inches and at bottom, not more than 36 inches. Portion 60 feet long on top, 30 feet long on bottom and 20 feet high was carried away.—*R. E. Thompson.*

Old Wooden Conduit Uncovered. Eng. News-Rec., 101: 281, August 23, 1928. Sections of wooden conduit recently uncovered in Chicopee, Mass., are estimated to have been in ground at least eighty years. Conduit was in sections averaging 7 feet long, with bore of about 4 inches. It was constructed of chestnut logs, roughly hewn on outside. Sections appeared still serviceable. Portions of line had been replaced with iron pipe of smaller bore, connections being made by enlarging openings in wooden pipe and driving iron pipe into place.—*R. E. Thompson.*

Jadwin Flood Control Plan Approved by Special Engineering Board. Eng. News-Rec., 101: 283–6, August 23, 1928. Unqualified approval of plan prepared by Major General EDGAR JAWDIN, Chief of Engineers, United States Army, for control of floods in Mississippi Basin has been given by board of

three appointed by President Coolidge to study plan in comparison with one prepared by Mississippi River Commission, and to make recommendations on all points on which the two reports differ. Board found that two plans were in close agreement in principle on at least 75 per cent of actual construction necessary and in partial agreement on remaining 25 per cent.—*R. E. Thompson.*

Several Cases of Saturnine Intoxication of Unusual Origin in a Family of Farmers. Rigot and Emeric. Ann. méd. légale criminol. police sci., 7: 370–3, 1927. From Chem. Abst., 22: 1001, March 20, 1928. Intoxication was traced to lead pipe, 100 meters long. Analysis of tap water showed: total solids at 100° 0.250, ignited solids 0.170, hardness 13° (French), organic matter 0.0009, small amounts of sulfate and lime, chloride a trace, lead 0.004 gram per liter.—*R. E. Thompson.*

Double Recorder for Testing Flue Gas. Max Berger. Apparatebau, 40: 9–10, 1928. From Chem. Abst., 22: 1028, March 20, 1928. Apparatus is based on frictional resistance to flow through capillary tubes, and records carbon monoxide and dioxide.—*R. E. Thompson.*

Drainage and Effluents from Gas Works. E. Jones. Gas J., 177: 516–8, 1927. From Chem. Abst., 22: 1029, March 20, 1928. Nature of effluents from gas works described, together with methods of treatment of effluents from ammonium sulfate plants before admission to sewer, either by removal of phenols from ammonia liquor by means of benzene, or by subsequent bacterial treatment of effluent. By Bailey process, toxicity of effluent was reduced by evaporation of effluent and "devil" liquors in two troughs arranged in by-pass to main flue, chimney gases being blown through liquors. Efficiency of purification, on basis of oxygen absorption test, was 33.5 per cent.—*R. E. Thompson.*

The Wastes of the Paper Pulp and Paper Mill Industries. J. B. C. Kershaw. Chem. and Ind., 46: 1173–5, 1197–9, 1927. From Chem. Abst., 22: 1040, March 20, 1928. Notes on sources of waters and on recent patents dealing with their utilization. Use of ferric chloride and lime for purifying waste sulfite liquors is mentioned.—*R. E. Thompson.*

Dissolution of Lead by Water in Pipes. A. Farine. Schweiz. Chem-Ztg., 1927, 29–32. From Chem. Abst., 22: 1082, April 10, 1928. Under similar conditions, distilled water saturated with air, distilled water containing (*a*) air and carbon dioxide, (*b*) air and sodium bicarbonate, (*c*) air, sodium bicarbonate and free carbon dioxide, when passed at fixed rate through tube packed with lead shavings, dissolved, respectively, 110, 10.5, 0.6 and 1.0 p.p.m. of lead. It follows that sodium bicarbonate exerts strong protective action which is less in presence of free carbonic acid. Results may be explained by physio-chemical considerations, which indicate that in presence of insoluble lead carbonate the concentration of lead dissolved is directly proportional to that of carbonic acid and inversely proportional to square of concentration of bicarbonate.—*R. E. Thompson.*

Electrochemical Corrosion of Iron by Cinders. Kurt Baum. Gas- u. Wasserfach 71: 10–11, 1928. From Chem. Abst., 22: 1125, April 10, 1928. Cinders in filled ground cause corrosion of iron due to setting up of electric cells. The greater the sulfur content of the cinders, the greater the corrosion. Cinders are not suitable for reinforced concrete as rapid electrolytic corrosion of reinforcing material results. Corrosion of boiler grates may be increased by E.M.F. set up by molten cinders in contact with solid cinders. Irons containing chromium (rustless) appear to be excellent for grates.—*R. E. Thompson.*

Corrosion Phenomena Which Occur in Centrifugal Pumps. E. Blau. Kohle u. Erz., Nos. 3–4, 49–53, 1927; Chimie et industrie, 18: 612, 1927. From Chem. Abst., 22: 1128, April 10, 1928. Discussion of nature and causes of corrosion in centrifugal pumps and of methods of prevention.—*R. E. Thompson.*

Intercrystalline Brittleness of Lead. Otto Haehnel. Z. Metallkunde, 19: 492–6, 1927. From Chem. Abst., 22: 1128, April 10, 1928. Failures occurring in lead cable sheaths are due to brittleness associated with formation of large crystals, and caused by mechanical vibration. Frequency of vibration, mechanical stresses, and temperature are contributing factors. Effect of temperature is small below 100°, and even with temperatures up to 260°, equilibrium is reached after a time and without mechanical vibration little effect would be produced. Addition of 1 per cent antimony or 3 per cent tin greatly diminishes velocity at which metal becomes brittle.—*R. E. Thompson.*

Recent Research in the Field of Drinking Water Purification. J. Tillmans. Z. angew. Chem., 40: 1533–9, 1928. From Chem. Abst., 22: 1200, April 10, 1928. Removal of iron from water depends largely on oxidation potential of ferrous ion. Formation of ferri-hydrosols depends on pH and not so much on concentration of dissolved electrolytes. It was found that aëration was enough to oxidize all iron at proper pH. Removal of manganese depends mostly on absorption of its salts on manganese dioxide and is greatly improved by layer of manganese dioxide on filter bed. Amount of iron dissolved from pipes is proportional to pH and to diameter. Rusting of such pipes is due to oxygen present; hence is proportional to dissolved oxygen. Action of carbonic acid in rust retarding was studied and theory is proposed.—*R. E. Thompson.*

The Diurnal Variation of the Gaseous Constituents of River Waters. Part III. R. W. Butcher, F. T. K. Pentelow and J. W. A. Woodley. Biochem. J., 1928, 22: 1035–47. From Bull. Hyg., 4: 27, January, 1929. The authors here record further results of their experiments on the Rivers Lark and Itchin during September to December for determining variations in dissolved oxygen, ammoniacal nitrogen, and pH values. The principal factors governing oxygen content of a river seem to be the quantity and type of plant life present, which is controlled by the season and the prevailing actinic conditions. Oxygen saturation figures for River Lark fall lower than those for the Itchin because of sugar beet factory wastes. The results give information on the relative effects upon oxygen saturation of river waters of photo-synthesis, respiration,

temperature, and absorption from air. Ammoniacal nitrogen curves indicate that the commencement of the maximum ammoniacal nitrogen follows the beginning of the period of minimum oxygenation and vice versa.—*Arthur P. Miller.*

A Simple Method of Chlorinating Water in Camp. E. W. Wade and H. P. Cavendish. Jour. Royal Army Medical Corps, **51**: 4, 285, October, 1928. The article describes treatment of an army water supply taken from a polluted stream in India. Apparatus consists of an 8-gallon can containing a 3 ounce to the gallon solution of chlorinated lime which is fed into the inlet of a 20,000 gallon elevated tank at point of discharge while tank is filling. Feed is regulated by a drip cock adjusted to the rate of 1 gallon solution to 2500 gallon water. One-half hour detention period is provided.—*A. W. Blohm (Courtesy U. S. P. H. Eng. Abstracts).*

Sterilization of Water Bottles by Means of the Lelean Sack. A. C. Hammond Searle. Jour. Royal Army Medical Corps, **51**: 4, 287, October, 1928. The Lelean sack disinfector in appearance closely resembles a sailor's canvas duffle bag, with the exception of a small opening in the wall of the bag near the base through which live steam is introduced. Sack used in this experiment was 4 feet in length and 20 inches in diameter. It is used "as is" for the steam disinfection of blankets, clothing, etc., and when fitted with circular racks; milk bottles and army water canteens may be disinfected. Optimum results are obtained by inverting the bottles and canteens. Articles to be sterilized are packed tightly in the sack. Sack is then suspended inverted, connected with the steam line and sealed by placing a blanket in the bag next to the draw string opening.—*A. W. Blohm (Courtesy U. S. P. H. Eng. Abstracts).*

Hydrogen Ion Concentration of the Water of Lake Geneva. W. H. Schopfer. Arch. sci. phys. nat., **8**: 22–5 (1926); Biol. Abstracts 1, 17. Chemical Abst., **22**: 16, 3007, August 20, 1928. Hydrogen ion determinations are given for water samples taken in series of depths at four stations in Lake Geneva on four dates. Surface readings varied between 7.6 and 7.85; bottom readings, between 7.2 and 7.65. Hydrogen ion figures on Lake Lioson and Lake Chaussey also are given. Brief comparisons are drawn between lakes of Wisconsin and of France.—*A. W. Blohm (Courtesy U. S. P. H. Eng. Abstracts).*

Determination of Gases Dissolved in Water. A. DeSalles Teixeira. Rev. Brasil. med. farm., **4**: 86–9 (1928). Chemical Abst., **22**: 16, 3007, August 20, 1928. A modification of A. Florence's apparatus is described, which permits the absorption of gases and withdrawal of the absorbing solutions without disturbing the vacuum. The burette is provided at its upper end with a stopcock and funnel, closed by a stopper with capillary. The lower end is connected with a bulb, and the latter with the flask and the Hg container, both connections being made by three-way cocks. The flask of known volume is filled with water, bulb and burette are evacuated and the water is boiled out. After the total gas volume is read the KOH, pyrogallol and $CuSO_4$ or $CdSO_4$ are introduced in the proper sequence through the upper stopcock.—*A. W. Blohm (Courtesy U. S. P. H. Eng. Abstracts).*

Operation of a New Filter Plant. J. Clark Keith. Canadian Engineer, 54: 17, 467, April 24, 1928. A description of the Essex Border Utilities Commission filter plant at Windsor, Ont., and discussion of results obtained during twelve months' operation. The works were put in commission in May, 1926, serving nine communities with a combined population of 110,000 people. The plant, which has a nominal capacity of 21 m.g.d., consists of a 48-inch steel intake in the Detroit River, revolving screens, coagulation basins of the "round-the-end" type and 10 rapid sand filters. On two occasions, frazil ice has formed on the mouth of the intake, almost cutting off the supply. Delivery of water into the intake, placing it under pressure, gave immediate relief, provision having been made for such manipulation of the pumps when the plant was constructed. The daily average pumpage during 1927 was 11,833,000 gallons. The total cost of filtration during the year was $30.15 per million gallons, including all fixed and operating charges.—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abstracts*).

Bacteriological Examination of Water. Anon. Jour. Royal Army Medical Corps, 51: 4, 278, October, 1928. This lengthy editorial contains a valuable history of attempts to differentiate between members of the coli aerogenes group. It includes results of investigations recently conducted by Houston in London, Hicks in Shanghai, J. Taylor, Clemesha, Thompson, Salle and others. The article describes Salle's method for differentiating between *B. coli* and the *aerogenes* group by measuring total acidity under controlled conditions.—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abstracts*).

Utilization of the Physical Properties of a Water for the Estimation of its Quality. F. Dienert. Revue d'Hygiene et de Medecine Preventative, 50: 881–93, 1928. The physical properties of a water are its taste, odor, color, turbidity, temperature and fluorescense. These physical properties can be utilized in estimating the quality of a water. Distilled water is said to have a blue color. Organic matter imparts yellowish and reddish tints to water. River waters are green because they contain yellow coloring matters mixed with blue. Spring waters, if pure, are generally colored blue. Contamination with surface waters may be detected by observing the color of such waters. Changes in turbidity and temperature may be correlated with variations in the bacterial quality of a water. Surface waters may contain humic compounds derived from the decomposition of vegetable matter, and some of these compounds are fluorescent. Waters which do not contain *B. coli* are never fluorescent. Fluorescence is determined by evaporating 500 ml. of water down to 20 ml., followed by filtration through a filter paper which has previously been treated with hypochlorite to destroy fluorescent materials. The filtered water is compared with a solution containing known amounts of fluorescein, using a Tyndall meter.—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abstracts*).

Disposal of Industrial Wastes. F. W. Mohlman and A. J. Beck. Ind. Eng. Chem., 21: 205–210, 1929. This paper presents detailed investigations of wastes from a corn products refinery and from a paint and dye works. It is shown that successful disposal of industrial waste is simplified by coöpera-

tive study with the technical staff of the factory and that changes in the production methods may not only eliminate excessive waste, but also make available valuable products. In relation to the corn waste, original testing station experiments indicated that a waste equivalent to a population of 400,000 people was being discharged into the canal involving potential outlay of about $2,900,000 for necessary treatment works. Subsequent coöperative study has made possible recovery of the solubles, reducing waste condition to an equivalent population of only 50,000. It is expected by additional study to reduce this figure to 20,000, or less, thereby making possible a much smaller treatment plant. The paint and dye waste resolved itself into a study of the toxic effect of arsenic, copper, and acid upon the operation of a combined Imhoff-activated sludge plant, it being found that these wastes reduced the capacity of such treatment works by about one-half. The acid wastes produced the greatest imhibition of digestion. The total volume of this (acid) waste, about 2 m.g.d., was discharged into the river in batches, giving rise to very acid conditions. "After consideration of the magnitude of any neutralization and the expense imposed, it was decided to try the effect of equalization of acid discharge" by the installation of an 18,000-gallon balancing tank, thereby possibly bringing the pH of the river to 6.4 or higher. It is expected that this process will eliminate partial neutralization of the waste. It is planned to remove the Cu from the waste by the use of Fe filings. This paper shows the economic value to the community and to the manufacturer of separation of the various types of wastes from a given plant, *in the plant, with separate treatment for each,* rather than promiscuous discharge into the most convenient sewer.—*Edward S. Hopkins.*

Chemical Treatment of Trade Wastes. Foster Dee Snell. Ind. Eng. Chem., 21: 210–13, 1929. This paper presents a study upon the treatment of wool washings. The process suggested is a combination of acid cracking of the wool washings with subsequent alum coagulation which gives an effluent of sufficient clarity to discharge into a stream. The alum is recovered, degras obtained, and some nitrogenous fertilizer made as by products. The amount of waste water treated was about 45,000 gallons per day at a cost for purification of about $50 per 1000 gallons. It is estimated that the sale value of the by products together with the alum recovery equals the expense of clarification. —*Edward S. Hopkins.*

Treatment of Industrial Waters from Paper Mills and Tannery on Neponset River. Almon L. Fales. Ind. Eng. Chem., 21: 216–21, 1929. The methods of preventing excessive pollution of the river by wastes discharged from two paper mills and a tannery are outlined. These wastes varied from 1 to 5 m.g.d. for each mill and had a volume of about 1 m.g.d. from the tannery. The paper mill wastes are settled with approximately four hours detention and the effluent passed through sand or coke filters which discharge into the river. The tannery wastes are similarly settled and the effluent filtered through sand, then passed into the river. The sludge has value as fertilizer. During the colder months, or during times of flood, the river does not become objectionable, so that purification is practiced during the late spring, summer, and

autumn which correspond to the time both of low stream flow and of warmer weather. During very low water the river is treated with sodium nitrate, thereby giving available oxygen for reduction of offensive odors. This is an excellent paper with many cuts and photographs.—*Edward S. Hopkins.*

Chemical and Biological Correlations in a Polluted Stream. WILLEM RUDOLFS. Ind. Eng. Chem., 21: 256–8, 1929. These studies show an apparent direct relation between the amounts of ammonia present and the bio-oxygen-demand of the Raritan River. As would be expected, oxygen depletion was greater in summer than in winter. Two small dams 4 to 6 feet high, gave slight increase in aëration and oxygen-content. A direct relation between bio-oxygen-demand and plankton was observed. Increase in oxygen-content was noted after rains.—*Edward S. Hopkins.*

Stream Pollution Control Activities in Wisconsin. L. F. WARRICK. Ind. Eng. Chem., 21: 261–3, 1929. With the increasing use of the automobile for recreational purposes, there has come an increasing demand for the protection of lakes and streams against pollution, from a sanitary and aesthetic sense. The control of such work is vested by the State of Wisconsin in a committee with the State Department of Health as the administrative agency. A program of coöperation with industries and municipalities has been adopted.—*Edward S. Hopkins.*

The Problem of Dilution in Colorimetric Hydrogen ion Measurements. EDNA H. FAWCETT and S. F. ACREE. J. Bact., 17: 163–203, 1929. A study is presented describing more exact colorimetric measurements of pH value for *very dilute* solutions. This is accomplished by the use of indicator solutions adjusted to approximately the same pH value as the material under test. This paper is of value in relation to bacteriological and water research problems, but has no bearing upon routine pH plant control.—*Edward S. Hopkins.*

A Survey of Thyroid Enlargement in Tennessee. ROBERT OLESEN. U. S. Public Health Reports, 44: 15, 865–897, April 12, 1929. A section of this article entitled **Endemic Goiter and Drinking Water,** includes the following: "In all probability endemic goiter is due, in part at least, to the absence from, rather than the presence in, water of a specific substance which normally aids in maintaining thyroid equilibrium. Food-stuffs as well as water should be considered." There appears to be no relationship between the amount of endemic goiter and the sources, treatment, and ultimate safety of public water supplies in Tennessee. While a slightly larger incidence of endemic goiter exists among users of chlorinated water in Tennessee, than among consumers of unchlorinated water, the incidence is slightly greater among users of unchlorinated water in Oregon.—*R. E. Noble.*

An Underground Water Research at Big Spring, Texas. JOHN B. HAWLEY. Proc. Am. Soc. Municipal Improvements, pages 257–62, 1928–1929. This paper discusses an investigation of the geology of the land in the vicinity of Big Spring as an aid in determining the probable amount of water that may be obtained from underground.—*John R. Baylis.*

Municipal Water Supply, Amarillo, Texas. M. C. Nichols. Proc. Am. Soc. Municipal Improvements, pages 263–4, 1928–1929. The author gives a brief discussion of the new well water supply for Amarillo.—*John R. Baylis.*

Bacterial Aftergrowths in Water Distribution Systems. John R. Baylis. Proc. Am. Soc. Municipal Improvements, pages 265–6, 1928–1929. In a number of cities there is an increase in the *B. coli* content of the water after passing through the distribution system. Where there is no possibility of pollution, the increase should not be considered alarming. Much of the increase probably can be attributed to the bacteria growing on dead microörganisms that gained entrance to the mains from open reservoirs.—*Frank Hannan.*

Soil Survey Cuts Cost in Preventing Pipe Line Corrosion. W. T. Smith. Chem. and Met. Eng., 36: 3, 137–8, March, 1929. A natural gas pipe line 340 miles in length has been laid from Amarillo, Texas, to Denver, Colorado. For part of the distance the line is 22 inches in diameter and for the balance, 20 inches. The degree of protection given was based largely upon the estimated corrosiveness of the soil. Composition of the soil and topography, considered independently, are inadequate; but when considered together, offer possibly the best method of predicting the corrosiveness of the soil. The procedure included sampling the soil at intervals in no case greater than 1 mile and at shorter intervals where surface conditions showed marked changes. Three hundred sixty soil samples were collected. Both the samples and soil extracts were analyzed. The relative activity of the soils upon steel has been confirmed by various laboratory tests and by the opinions of men experienced in pipe line construction.

Representative analyses of water extracts of soil samples (parts per million)

	EXCELLENT	GOOD	FAIR	POOR	BAD	VERY BAD
Total solids	64	151	405	320	692	3,333
Silica	10	15	210	35	25	13
Calcium	32	32	31	49	91	294
Magnesium	3	18	27	24	120	603
Iron and aluminum oxides	5	20	15	30	10	10
Sulfate	0	42	90	144	166	2,259
Carbonate	3	0	0	0	0	0
Bicarbonate	2	8	8	16	10	8
Chloride	0	9	21	15	260	127
Sodium and potassium	9	7	3	7	10	19

Certain trends of some of the items are definite, and others appear inconsistent with the classification made. The analyses are very good illustrations of the necessity of basing estimates of corrosion on all possible information rather than upon some such single item as sulfate content, or hydrogen-ion concentra-

tion. Based on chemical rating of the samples of soil along the route, the classification was as follows:

CLASSIFICATION	PERCENTAGE
Excellent and good	16.39
Fair and poor	53.33
Bad and very bad	30.28

Considering chemical composition only, 16.39 per cent of the line would have received no coating, 53.33 per cent a light coating, and 30.28 per cent a heavy coating. The classification was revised to make allowance for topographical conditions. In crossing drains, at dry washes, under dry creek beds, in sinks, in any locality subject to intermittent moisture, and in cultivated lands where the chemical analysis indicated the necessity of light protection, heavy was specified. Pipe located in rocky or sandy soil, rated "excellent," or in very sandy soil, rated "good," was allowed no protection if the ground formation gave evidence that it would be consistently dry. The following table shows the specified protection:

PROTECTION SPECIFIED	BASED UPON COMPOSITION ALONE	BASED UPON CHEMICAL COMPOSITION AND TOPOGRAPHY
	per cent	*per cent*
No coating	16.39	7.25
Light coating	53.33	42.29
Heavy coating	30.28	50.46

The line would have been given heavy protection all the way had there been no means of predicting the corrosiveness of the soil. The saving was $197,380.—*John R. Baylis.*

The Water Supply of Detroit, Michigan. F. H. STEPHENSON. Proc. Am. Soc. Municipal Improvements, pages 267–71, 1928–1929. The distribution system comprises about 3400 miles of pipe from 4 to 72 inches in diameter. The Department lays nearly all the pipe with its own forces. To make up for lack of reservoirs, three 1.5-million-gallon capacity elevated steel storage tanks have been built, and another is under construction. A new river intake and tunnel are being constructed, and plans are under way for a new filtration plant.—*John R. Baylis.*

Engineering Geology as Applied to Location of Tunnels and Dams. FRANK E. FAHLQUIST, Boston Soc. Civ. Eng., 16: 1, 1–13, January, 1929. For exploratory work the surface topography should be divided into two main divisions: (1) the ground moraine, of shallow depth, composed of the unmodified glacial drift deposits, and (2) the deeper, modified and unmodified, glacial drift

deposits. Consideration must be given to the abrasive and plucking action of the ice sheet such as the local direction of glaciation, geologic structure, and orientation of the valleys with respect to glaciation. Attempts should be made to trace out the preglacial drainage courses as far as they might affect the nature of the work. N. L. HAMMOND. Borings for the Metropolitan District Water Supply were contracted for at $4.35 per linear foot. Diamond drills were used and the diamond loss kept low due to drilling being stopped immediately upon penetration through a boulder and not being allowed with the bit in sand. Holes were placed longitudinally along the axes of the valleys to determine the probable slope of the rock floor. The average depth of all holes above bedrock was 87 feet and the maximum depth drilled was 164.58 feet. One hole required twenty-two days of actual drilling with an average rate of 6.1 feet per day. All machinery used was gasoline-driven. Dry samples were taken about every five feet.—*J. F. Pierce.*

Water Supplies from Wells and Springs. Anon. Water and Water Engineering, 31: 362, 53, February 20, 1929. Brief review of a memoir entitled "Wells and Springs of Sussex," from a series now being prepared and issued by the Geological Survey of England and Wales. The pamphlet is, in a sense, technical, but most of it is prepared so as to be understandable by the layman. The data are selected principally from the standpoint of underground water supplies, but much information is also given on the general geological structures and stratigraphical details. Sussex has a fluctuating seasonal water demand due to the influx of visitors and tourists and is likewise noted for the large number of wells and boreholes in isolated areas intended for furnishing water to well-to-do residents. The water-bearing strata in this section are not always dependable; some have been found to yield an unsatisfactorywaterand others have given out after a short period of use. Useful additional information is given on chalk hydrology.—*Arthur P. Miller.*

New Filtration Plant for Barnsley Corporation. Anon. Water and Water Engineering, 31: 362, 55, February, 20, 1929. Additional filtering facilities were required by the Barnsley (England) corporation to handle increased water supply. The old filters were of the slow sand type, but on account of space limitations, it was decided to design the new ones on the rapid basis. A building to house the new filters was available on the property and tenders were invited on the understanding that the new filters had to be fitted into the existing building and its back yard. The water from the old filters was treated to avoid action on lead by passing half of it over a bed of limestone and the other half over chalk. In the revamping of the old plant when the new was built, arrangements were made to treat all water from the older plant with lime. The new plant includes four filter beds, each of $\frac{1}{4}$-m.g.d. capacity, chalk and alumina mixing tanks, two line saturator tanks, feeding gear for all chemicals, chemical storage, air compressor, and other accessory pumps and meters. The method of operating the plant is given in some detail. There appears to be nothing unusual in the system followed.—*Arthur P. Miller.*

Something about Pumps and Pumping. F. J. GARLAND. Water and Water Engineering, 31: 362, 60, February 20, 1929. Pumping in some form has always been and probably always will be necessary to man's existence because without water man could not exist long. The story of the inception of the construction and installation of pumping machinery dates from the primitive windlass and makes an interesting narrative. The author lists the numerous pump developments within the last 150 years and discusses the circumstances and conditions under which they are applicable. In his discussion, he cites some examples of pumping machinery which were classics in their day. The principal demands made upon pumps today are durability, reliability, and efficiency. Opinion may differ as to which of these is most important but the author feels that in public service stations, reliability should be given first consideration.—*Arthur P. Miller*.

Egypt's Water Supply. Anon. Water and Water Engineering, 31: 362, 63, February 20, 1929. The Egyptian government has decided to raise the Assouan dam on the Nile River 23 feet. This great dam retains the flood waters of the river for irrigating purposes. It is about 6400 feet long and consists of two parts; a solid masonry dam 1800 feet out from the east bank and a masonry dam 4600 feet out from the other bank which part contains the sluices and lock. A peculiarity of this dam is that it had to be designed so that the entire flow of the river could be discharged through sluices; no water being permitted to pass over the dam. An International Technical Commission reports that conditions of foundation and the present structure are satisfactory to receive the superimposed additional elevation.—*Arthur P. Miller*.

Water Pollution Research. Anon. Water and Water Engineering, 31: 362, 64, February 20, 1929. The 1927–28 annual report of the Department of Scientific and Industrial Research (England) outlines the accomplishments for the year. Arrangements have been made for the preparation of monthly sumaries of the related literature which are distributed to other departments, universities, and like institutions; contacts have been established with other research Committees and interested societies; a proposal to undertake a biological and chemical survey of the River Tees is under discussion; an investigation of the zeolite softening process has been initiated; and plans have been completed for a biological investigation of the activated sludge process. Prior to the formation of the Water Pollution Research Board, some experimental work on the purification of the effluent from a sugar beet factory had been done at the Rothamstead Experimental Station. A mass of valuable information was obtained but the experiments could not be considered conclusive. Hence, arrangements have been made for the continuation of this work during the next season.—*Arthur P. Miller*.

Contamination of Water Supply at Nacogdoches, Texas, by Uncovered Tanks. R. G. UPTON, Southwest Water Works Journal, 11: 1, 17–18, April, 1929. Inspections and tests showed B. Coli contamination of supply due to uncovered catch basin at wells. Basin had area of 2500 square feet and was located less than 50 feet from the T. and N. O. and H. E. and W. T. tracks and at a little

greater distance from the city power plant, planing mills, two public roads, and several houses, including the section houses, all in the unsewered district of the City. Covering the reservoir resulted in improved quality.—*John H. O'Neill.*

Water Works Service Pipes; Newer Materials and Practice. Chas. H. Ade. Southwest Water Works Journal, 11: 1, 14–16, April, 1929. Paper read at 1929 Texas Waterworks Short School. The more general use of street paving has greatly increased the importance of proper materials for service connections and the need of using the best available materials. A strong argument is made for the use of heavy 85 per cent copper, hand ground, corporation and curb cocks, and of copper service pipe.—*John H. O'Neill.*

Considerations in the Design of a Waterworks System for a Small Town. Julian Montgomery, Southwest Water Works Journal, 11: 1, 18–20, April, 1929. The five major considerations in the obtaining of a waterworks system are: promotion, financing, design, construction, and operation. The article includes a short discussion of each item. *John H. O'Neill.*